Santa Clara Valley Water District
Water Resources Management Group
Hydrology and Geology Unit

QUALITATIVE GEOTECHNICAL REPORT

LOWER GUADALUPE RIVER PROJECT

by

Mohammad A. Khan, P.E., Assistant Civil Engineer II

Under the direction of

Abdullah D. Saah, P.E., P.H.
Supervising Engineer

Keith Whitman
Water Resources Management
Group Manager

James T. Ferguson
Executive Project Manager

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September, 1996

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I. INTRODUCTION

This report presents the results of a qualitative geotechnical investigation performed by the Santa Clara Valley Water District for the proposed Lower Guadalupe River Project in San Jose, California. The project plan is not finalized yet. The preliminary project plan (see Appendix A) indicates that the project will extend from north of Alviso Marina to highway I-880 in San Jose (from about Station 56+00 to 176+00). Reference stations are approximate, and in meters (i.e.: 176+00 = 17,600 meters). The final project may extend further downstream to the Bay (to about Station 0+00). Based on the conceptual plan, it is understood that the purpose of the project is to further increase the capacity of the Lower Guadalupe River to carry the 1% flood with a minimum of 0.9 meter (3 feet) freeboard. Several options are currently being considered to increase the channel flow capacity. One of the options is to increase the flow capacity by raising the existing levees to an elevation such that it maintains a required minimum freeboard during the peak flow caused by a 1% flood.

The purpose of this report is to identify the major geotechnical considerations for this project and provide qualitative geotechnical recommendations for preparing a feasible project plan. After finalizing the project plan, a quantitative geotechnical investigation will be conducted at the project site and a geotechnical report prepared for project design and construction. Based on the review of available construction plans, It is understood that levees were first built in 1963 along both banks of the river. Afterwards, both the river channel and the center lines of the levees were realigned and the old levees were raised in 1983. In 1995, as an emergency measure, both east and west levees from Southern Pacific Rail Road (SPRR) to Highway 101 (from about Station 70+00 to 145+00) were raised again to provide additional freeboard against a 1% flood. At the time of writing this report, as-built drawings for levees were not available. Based on discussions with Mr. Iwao Shintani, Mr. Tri Nguyen, and Ms. Rechelle Blank, it was understood that some modifications were made in the plans during construction of the levees in 1983, and significant modifications were made during the 1995-construction.

As outlined in a July 26, 1996, memorandum, the following scope of work was performed to prepare this report:

1. Review all available construction plans and cross-sections for the existing levees.
2. Conduct site reconnaissance.
3. Review all available fill material specifications and compaction test results for previous levee construction.
4. Review all geotechnical reports for previous levee design and construction.
5. Review crest elevation data and land subsidence records to evaluate the settlement behavior of existing levees.

II. DISCUSSION

1. Construction Plan Review: 1983 Construction Plans indicate that the old levees were realigned and raised by about 0.30 to 2.40 meters (about 1 to 8 feet). Typical cross-sections for the new levees were designed with 1.5:1 and 2:1 (horizontal to vertical) side slope ratios. 1995-Construction Plans indicate that from SPRR to Highway 101(from about Station 70+00 to 145+00) the previous levees were raised again by about 0.30 to 0.90 meter (about 1 to 3 feet). The purpose of the new levee raising was to provide a minimum of 0.45 meter (1.5 feet) freeboard against the 1% flood. This new levee raising was designed with six different cross-sections and side slopes to be used in different areas as follows:

    (i) A typical cross-section (Type 1) with no soil reinforcement, and having a crest width of 2.74 meters (9 feet) was designed with 1.5:1 outboard side slope and 1.75:1 inboard side slope

    (ii) A cross-section (Type 1A) with soil reinforcement, and having a crest width of 3.65 meters (12 feet)) was designed with outboard side slope stepper than 1.25:1 and inboard side slope equal to 1.75:1

    (iii) A cross-section (Type 1B) with soil reinforcement, and having a crest width of 4.27 meters (14 feet) was designed with both inboard and outboard side slopes equal to 1.75:1

    (iv) A cross-section (Type 1C) with soil reinforcement, and having a minimum crest width of 3.04 meters (10 feet) was designed with 0.75:1 outboard side slope and 1.5:1 or 1.25:1 inboard side slope

    (v) A cross-section (Typical Overlay Levee) having a minimum crest width of 3.04 meters, and a key in the outboard slope was designed with 1.75:1 inboard slope and 1.5:1 or 2:1 outboard slope

    (vi) A cross-section (Type 2) with an L-shaped concrete retaining wall (flood wall), about 1.06- meter (3.5 feet) high (see Appendix A).

The as-built drawings for 1995-constructions were not available at the time of writing this report. Since a significant modification has been made in the plans during construction of levees in 1995, the construction plans could not be compared with the actual field conditions. However, recent field visits have helped to observe the existing levee conditions and evaluate its performance. Following sections discuss the results of field reconnaissance; fill compaction tests; reviews of geotechnical reports; topographic surveys and evaluations of the past and future performance of levees from a geotechnical standpoint.

2. Site Reconnaissance: During the month of August, 1996, site reconnaissance was conducted in order to evaluate the existing levee conditions. Reconnaissance work included observation of both east and west levees and the surrounding topography from north of Alviso Marina to Highway- 880 (from about Station 53+00 to 176+00). Photographs were also taken of major surface features upon walking over the levees. Copies of photographs showing significant surface features are attached to this report in Appendix B. The reference stations shown in the photographs are determined in the field based on eye estimate only. The actual stations may vary.

Field observations indicate that the outboard slope of the east levee downstream of Alviso Marina (from about Station 60+00 to 61+00) has failed. Since there is no slope protection measure applied in this area, the slope failure appears to have been caused by surface erosion and wave action from the adjacent aeration pond (see photographs in Appendix B). This erosion process is ongoing and may further weaken the levee. Therefore, the failed area of the east levee north of Alviso Marina will need immediate repair to prevent progressive failure.

Recommendations for interim levee repair and slope protection are provided in the next section of this report. Recommendations for new levee design and long term slope protection will be provided after the project plan is finalized and an additional geotechnical investigation is completed.

Observations indicate that dredge sediment from the adjacent aeration pond has been loosely dumped on the part of east levee further north of Alviso Marina (at about Station 53+00). This uncompacted, loose sediment fill is about 0.60 to 0.90 meter (about 2 to 3 feet) thick and has been broken into chunks of clay by dessication creating an uneven surface over the levee, blocking access to the levee further downstream. For future inspection and subsurface investigation along the east levee further downstream from about Station 53+00, this sediment fill will need to be removed, or levelled by pulverizing and recompacting it in order to provide access to vehicles, equipment, etc.

Observations also indicate that there exists a concrete rubble fill on the outboard side of the west levee near the Cargil Salt Pond, north of SPRR at about Station 67+00. This rubble fill is about 3.0 meter (10 feet) high, and blocks access to the west levee further downstream. For lack of access, inspection could not be conducted along the west levee downstream of about Station 67+00. To conduct future inspection and geotechnical investigation along the west levee further north of SPRR, the concrete rubble will need to be removed.

However, based on discussions with project team members, it is understood that the land properties and the levees north of Alviso Marina or SPRR may be owned by agencies other than the District. If so, the actual responsible agencies for land and levees in this area should be identified, and the District's effort to redesign or repair the existing levees should be coordinated with those agencies.

During inspections of levees from SPRR to Highway 237 (from Station 70+00 to 80+00), it was found that both east and west levees have been raised with much steeper slopes than they are originally shown in the 1995 construction plans. The average levee side slopes in this area appear to vary from about 1.25:1 to 1.5:1. It appears that due to minimum crest width requirement and limited land space available within the District's right of way, levees had to be raised with steeper slopes during 1995 emergency construction. However, it is understood that soil reinforcement with Tenax MS-550 fabric was used in some parts of the levees where construction with steep slopes were needed.

Observations indicate that both east and west levees between SPRR and Highway 237 have developed some cracks in several locations. These cracks are about 0.60 to 1.30 centimeter (1/4 to 1/2-inch) wide, and in some area it extends up to about 15 meters (50 feet) long (see photographs in Appendix B). Some cracks on the levees in this area appear to be simple tension cracks. Other cracks may have been caused by the differential crest settlement, excessive shrinkage of fill material, and inadequate soil compaction near the edges of the levee crest resulting in a localized failure of the steep slope. Because the as-built drawings for 1995 construction showing the actual locations of soil reinforcement are not available yet, the effectiveness of soil reinforcement in reducing the tension cracks and stabilizing the steep slope could not be evaluated reliably. However, experience indicates that such a steep slope (< 1.5:1) in a levee is generally not very stable.

Observation also indicate that both east and west levees from Highway 237 to Tasman Road (from Station 80+00 to 92+00) have also developed similar cracks. Some cracks in this area range from about 15 to 30 meters (50 to 100 feet) in length, and about 0.60 to 1.2 centimeter (0.25 to 0.5 inch) in width. The average side slopes of the 1995 levee raising appear to vary from about 1.3:1 to 1.5:1. The probable causes of these cracks have been discussed above.

Observations of existing east and west levees from Tasman Road to Montague Expressway (from Station 92+00 to 118+00) indicate that some cracks have been developed near the edges of the levee crest. The side slopes of 1995 levee raising appear to be ranging from 1.4:1 to 1.5:1 and there exist some unprotected open slopes from south of Tasman Road to near Hetch-Hetchy. From about Station 96+60 to 101+00, a concrete retaining wall (flood wall) about 0.60 meter (2 feet) high has been used at the outboard edge of the east levee for 1995 raising. The concrete wall appears to be effective in holding the new fill in this area. However, right before the beginning of the wall, a kind of geo-net (Strawmat S150) has been used as a slope protection measure on the outboard side of the east levee from about Station 96+00 to 96+20. This geo-net appears to be torn apart over the slope, and may not be effective in protecting the slope in the long run.

Field inspections indicate that a portion of the inboard side slope of the east levee immediately north of Montague Expressway (at about Station 118+00) has failed due to localized sliding. This failed slope, if left unrepaired, may cause progressive failure on the levee when it becomes saturated during the rainy season. Also, a portion of the inboard side slope of the west levee immediately south of Montague Expressway (at about Station 118+50) has been lined with concrete. This concrete-lined slope appears to be in stable condition at the present time. However, localized shallow slide may occur on the upper unlined slope which appears to be steeper than the lower concrete slope.

Similarly, field observations from Montague Expressway to Trimble Road (from Station 119+00 to 136+00) indicate that both east and west levees have also developed some tension crack and are left with unprotected, open slopes in several locations. These open slopes are eventually subject to erosion. Moreover, several animal burrows along the levees have been observed in this area. These animal burrows may cause progressive failure in the levees when they become saturated with rain or flood water.

Observations of both east and west levees from Trimble Road to Highway-101(from Station 137+00 to 145+00) indicate that there is no major cracks on the levees. However, there exists some unprotected steep slopes along the levees of this area which may erode over the course of time. It was also observed that a portion of the inboard slope of the east levee at about Station 138+00 has been covered with sand bags to stabilize the slope and from about Station 144+00 to Highway-101 the inboard slope is covered with a concrete lining.

The levees from Highway-101 to Highway-880 (from Station 146+00 to 176+00) appear to be in good condition except for some tension cracks on the east levee near the Station 148+00 (near the airport long-term parking). The inboard slope of the west levee immediately south of Highway-101 has been covered with sand bags to stabilize the slope in this area (see photographs in Appendix-B).

3. Fill Material & Compaction: Review of available fill compaction results indicates that generally two distinct types of fill materials were used in 1995 levee raising operations. Type A (a clayey soil used beneath the finished grade) and Type-B (a Class II Aggregate Base, ***AB***, used on top of the clayey fill to prepare the finished grade). The field density test results show that most of the fill was compacted to more than the minimum required relative compaction of 90%. However, the results of Atterberg's Limit Tests indicate that the Plasticity Index (***PI***) of the fill soil mostly ranges from about 31 to 39, and the Liquid Limit (***LL***) mostly ranges from about 51 to 74. Soils with such a high ***LL*** and ***PI*** are classified as Highly Plastic Clay, or Fat Clay (***CH***), and are generally unsuitable for use as fill materials. Experience shows that clayey soils with ***LL*** > 50%, and ***PI*** > about 25, when used as fill, usually undergo excessive shrinkage or swelling, and often they are difficult to compact. As such the District's specification limits the ***PI*** of levee fill to range from a minimum of 10 to a maximum of 25. Thus it appears that the soil used as levee fill did not meet the specification standards. Copies of the District's specifications, the Field Density test, and Atterberg's Limit Test results are attached to this report in Appendix C.

Moreover, field observations indicate that the fill materials near the edges of the levee crest were not adequately compacted. There remains a lot of loose fill near the edges which extends over the levee slopes. Under general conditions, a big compactor machine can not achieve the desired compaction near the open edges of the embankment crest for lack of confinement. When a big machine is passed near the edges, it just pushes the fill aside over the slope. Usually a small portable hand compactor is used to compact the finished grade soil near the edges and the slopes are trimmed from the edges to remove the loose material.

4. Previous Geotechnical Reports: One report was prepared by Woodward-Clyde Consultants (Dated January 13, 1993) for 1983 levee design between Alviso Marina and the SPRR (from about Station 60+00 to 70+00). The report indicates that the subsurface soils in this area are typically old levee fill about 1.2 to 4.5 meter (about 4 to 15 feet) thick underlain by a deposit of soft Bay Mud and stiff clay. The thickness of Bay Mud varies from about 1.2 to 10 meters (about 2 to 34 feet). This soft Bay Mud is underlain by inter-bedded layers of stiff clay and medium dense to dense silty sand or sand to the depths of boring. Groundwater elevation in this area was reported to be ranging from about -1.5 to +1.5 meters (about -5.0 to +5.0 feet). A copy of the site plan and an idealized soil profile from this report are attached in Appendix D of this report.

The 1983 Woodward-Clyde report estimated the ultimate levee settlement in this area to be ranging from about 5.0 to 50.0 centimeters (2.0 to 20.0 inches) and it would take about 1 to 25 years for 90% settlement to occur over different parts of the levee. Also, this report identified the evidence of seepage through the levees and recommended a possible seepage control measure to be applied during the levee construction. Based on engineering analyses, this report concluded that a liquefaction potential exists for the underlying saturated silty sand or sand, and a horizontal displacement of levee up to 0.30 to 1.5 meters (1.0 to 5.0 feet) was expected during a major earthquake resulting in a complete levee failure. However, for static stability, the 1983 levee between Alviso Marina and SPRR was recommended to be designed with a side slope ratio of 2:1. No geotechnical report was available for reviewing the existing subsurface soil conditions and levee design further downstream of Alviso Maria. However, the subsoil conditions further downstream are expected to be similar to that of the Marina area.

One geotechnical report was prepared by John V. Lowney & Associates in January, 1981, for design of the 1983-Levee from SPRR to Highway-237 (from Station 70+00 to 80+00). According to this report, the old levee materials consist of a mixture of sand, silt, and clay. The soils beneath the old levee consist of soft and compressible clayey silt (Bay Mud). The thickness of Bay Mud varies from about 0.60 meter (2.0 feet) at boring EB-8 located near Highway-237 (Station 80+00) to about 13.0 meters (43.0 feet) at boring EB-6 located near the Station 75+00. Pockets of organic matter were encountered within the Bay Mud. In addition, a layer of garbage consisting of paper and glass fragments was encountered in boring EB-1 located near the SPRR (Station 70+00). A copy of a site plan from the John V. Lowney & Associates report showing the boring locations is attached in Appendix-D.

At the time of drilling, free groundwater was encountered at approximately the mean sea level elevation and the surface water level in the river was reported to be fluctuating with tide through a height of about 1.80 to 2.40 meters (6.0 to 8.0 feet). Because of the varying thickness of underlying soft Bay Mud, the Lowney & Associates' report estimated the ultimate levee settlement to be ranging from about 0.90 centimeter (0.30 foot) near the boring EB-2 to about 0.70 meter (2.30 feet) near the boring EB-6. This amount of settlement was estimated to be completed in 85 years. The profile elevation data prior to 1995 levee raising show that the 1983 levee has already settled about 0.38 meter (1.50 feet) near the boring EB-6 (near Station 75+00) and about 0.24 meter (0.80 foot) near Highway-237 (near Station 79+00). Thus it appears that the actual levee settlement is consistent with the predicted settlement in this area.

Based on static and seismic stability analyses, this report recommended a levee design with 2:1 slope, and it concluded that localized sliding could occur in different parts of the levee during a major seismic event. However, the results of seismic stability analysis in this report show that the levee becomes unstable when the lateral ground acceleration exceeds 0.05g. Experience shows that during a major earthquake in California, the ground acceleration is usually much higher than 0.05g, therefore, the seismic stability and safety of levees will need to be further analyzed for the design of new levees. Geotechnical reports prepared by others indicate the presence of compressible Bay Mud under the levee in this area and estimate an ultimate levee settlement to be as high as about 0.90 meter (3.0 feet).

No geotechnical report was available for design of existing levees from Highway-237 to Montague Expressway (from Station 80+00 to 118+00). However, the District's survey records show that from 1983 to 1995 the existing levees in this area have settled about 0.20 meter (0.70 foot) near Station 96+00, to about 0.45 meter (1.50 feet) near Station 145+00. This indicates the presence of compressible clayey soils or Bay Mud under the levees in this area.

For previous levee design and channel excavation between Montague Expressway and Trimble Road (from Station 119+00 to 136+00), one geotechnical report was prepared by Joe Crosby & Associates in January, 1976. This report indicates that the subsurface soils in this area are typically composed of stiff to very stiff silt and clay and medium dense to dense sand and gravel to the depths of boring, about 10.0 meters (30.0 feet). Apparently these materials were deposited by streams draining the valley floor. Stiff silt and clay or dense sand and gravel are relatively incompressible and have a higher shear strength than Bay Mud. Consistent with the underlying soil conditions, levees in this area appear to be in a more stable condition and over the years they appear to have settled much less than that over Bay Mud.

No geotechnical reports were available for previous design of levees from Trimble Road to Highway-101 (from Station 137+00 to 145+00) and from Highway-101 to Highway-880 (from Station 146+00 to 176+00). The existing levees appear to be in a good condition without any significant settlement or slope instability along most of the levee alignments. However, the 1995 profile elevation data show that the east levee near Highway-101 (near Station 144+00) has settled about 0.67 meter (2.20 feet) within a 12-year period from 1983 to 1995. This amount of consolidation settlement appears to be inconsistent with the expected subsurface soil conditions in this area. One could conclude that the previous land subsidence in this area caused by groundwater lowering may have contributed to this high levee settlement. Additional survey data reviews and geotechnical investigations would be needed to evaluate the actual cause of such a high levee settlement in this area.

5. Review of Crest Elevation & Land Subsidence Data: At the time of writing this report, no crest elevation data were available for levees downstream of SPRR. Field observation indicates that the east levee north of Alviso Marina (near Station 60+00) has settled unevenly between the center and the edges. The actual amount of levee settlement could not be determined at the time of writing this report for lack of available survey records in this area. Because of underlying soft Bay Mud, a large amount of levee settlement is expected in this area.

Profile elevation data prior to 1995 levee raising show that both east and west levees from SPRR to Highway-101(from Station 70+00 to 145+00) have undergone a varying amount of settlement ranging from 0.20 to 0.67 meters (0.70 to 2.20 feet). Part of this levee settlement over the years could be attributed to the surrounding land subsidence caused by groundwater lowering.

Available land subsidence records near Gold Street show that the land has subsided only about 0.06 meter (about 0.20 foot) in this area over a period of 12 years from 1976 to 1988. The land subsidence in this area appears to have been stabilized. Also, the available survey records from a location near Lafayette Street show that the land has subsided about 0.90 meter (about 3.0 feet) over a period of 7 years from 1960 to 1967. A copy of the available land subsidence records is attached in Appendix D of this report. Additional land subsidence data beyond 1967 are not available for this location. Based on discussions with Mr. Gary Faler, other subsidence measurement locations are far away from the existing levee alignment, therefore, the limited land subsidence data are inconclusive at the present time for evaluating its effect on the settlement of levees. Some new stations for subsidence measurement have been established near the levees. Land subsidence records from these stations will be available in the future to evaluate its impact on the levee settlement.

III. CONCLUSIONS

On the basis of the above discussion, it is concluded that the raising of existing levees to increase the channel flow capacity is feasible from a geotechnical engineering standpoint, provided the recommendations presented in the next section of this report are incorporated in the project plan for final design and construction. The main geotechnical considerations for this project can be divided in to two categories: one category is the short-term consideration, and the other is the long-term consideration.

1. Short-Term Consideration: The short-term geotechnical considerations for this project are:

    (A) Repair of existing cracks, slope erosions, animal holes, and shallow slides; completion of topographic survey along the proposed levee alignment; clearing the existing concrete rubble and loose sediment fill from the levees downstream of SPRR and Alviso Marina.

    (B) Based on observations it is concluded that the present cracks, slope erosion, and shallow slides do not pose any immediate threat to the overall stability and safety of the existing levees. Since the project plan is not finalized yet, and the existing levees may have to be realigned for environmental mitigation measures, it is recommended that only minimum repair works be performed for the existing levees in order to prevent the progressive failure before a final design is completed. Specific recommendations for short-term levee repair are provided in the next section.

    (C) Field observations indicate that there exists some deep ditches at the toe of the outboard slope of the existing levees overlying the soft Bay Mud. These ditches will form water ponds and saturate the outboard levee slope during the rainy season making it a critical section for slope instability. A topographic survey map showing the elevations of the existing and proposed levee alignments and the surrounding land will be essential for proper investigation, settlement evaluation, and safe design of new levees. A topographic map may also be needed for land acquisition and environmental planning for this project, therefore, a topographic map should be prepared immediately for this project so that it would be timely available for planning subsurface investigation and new levee design.

    (D) As discussed in the previous section, the existing concrete rubble and the loose sediment fill should be removed immediately from the levees downstream of SPRR and Alviso Marina, so that there is access to the levees further downstream for field inspection and geotechnical investigation.

2. Long-Term Consideration: The long-term geotechnical considerations for this project are:

(A) The present consolidation characteristics of the underlying Bay Mud or soft clay and future settlement of levees including the effect of land subsidence.

(B) Static and seismic stability of new levees.

(C) Liquefaction potential for the underlying saturated silty sand or sand.

(D) Potential seepage through the pervious foundation soils

(E) Erosion control and slope protection measures.

The previous geotechnical investigation data appears to be incomplete and inadequate for design of the new levee. Additional geotechnical investigation and engineering analyses will be needed to address the long-term geotechnical considerations as mentioned above and to design the new levees for this project.

Based on field observations and discussions with other project team members, this project has several environmental constraints and limited space available for further levee raising within the District's right of way. Considering these limitations, conceptual recommendations for different design options have been provided in the next section for project planning.

IV. RECOMMENDATIONS

1. Existing Levee Repair:

Grading: In order to prevent the progressive failure in the existing levees, some repair works should be performed immediately in the area of major cracks, slope erosion, animal burrows, and shallow slides. First the area to be repaired should be cleared off the existing debris and weeds and stripped of the upper soil containing organic matter if encountered. Cleared weeds and debris should be removed from the site.

Following the clearing and stripping, the levee fill within the crack or slide area should be excavated and stockpiled in a suitable location for later use. All loose soils from the bottom of the excavation should be removed. The exposed sub-grade should be scarified to a depth of at least 15.0 centimeters (6.0 inches), moisture conditioned to 2 to 3 percent above optimum water content, and recompacted to at least 90 percent relative compaction based on ASTM D1557 standard. Following sub-grade preparation, the ground surface should be kept moist to prevent excessive shrinkage due to moisture loss.

Fill Materials: Before placement the excavated fill materials and any imported fill should be tested for suitability as levee fill. Generally, the fill should be free of organic matter, deleterious substances, debris, and rocks or lumps larger than 10.0 centimeters (4.0 inches) in greatest dimension. No more than 15 percent of the rocks or lumps should be larger than 6.0 centimeters (2.50 inches). In addition to these general requirements, the levee fill should have a low expansion potential. For the purposes of this report, a low expansion potential is defined by a liquid limit (*LL*) of less than 50 and a plasticity index (*PI*) below 20.

Placement and Compaction: Fill should be placed in lifts that do not exceed 20.0 centimeters (8.0 inches) in loose thickness and compacted to at least 90 percent relative compaction. Prior to compaction, sandy soils should be moisture conditioned to near optimum water content, and clayey soils moisture conditioned to about 2 to 3 percent above optimum water content.

Since the existing levees may need to be broken down, and realigned for environmental mitigation measures, no erosion control or slope protection measures are recommended at this time until the project plan is finalized and the new levees are constructed.

2. Design of New Levees: Due to limited space available within the District's right of way and the minimum crest width requirement, most of the existing levees have been constructed with much steeper slopes than it was originally recommended for a stable levee design. Additional raising of the existing levees in a conventional way would require a flatter slope for long-term stability. Considering the limited space available, and the associated environmental constraints, it is recommended that the following conceptual design options for planning purposes be considered:

   Option- 1: Raise the existing levees vertically by using concrete retaining walls (flood walls) along both edges of the crest with a mutual tie-back system. Backfill the walls with a suitable fill material, and provide periodic maintenance services as needed.

   Option-2: Raise the existing levees as required with steep slopes to provide a minimum crest width. Provide appropriate soil reinforcements to stabilize the steep slopes for long-term performance, and provide periodic maintenance services as needed.

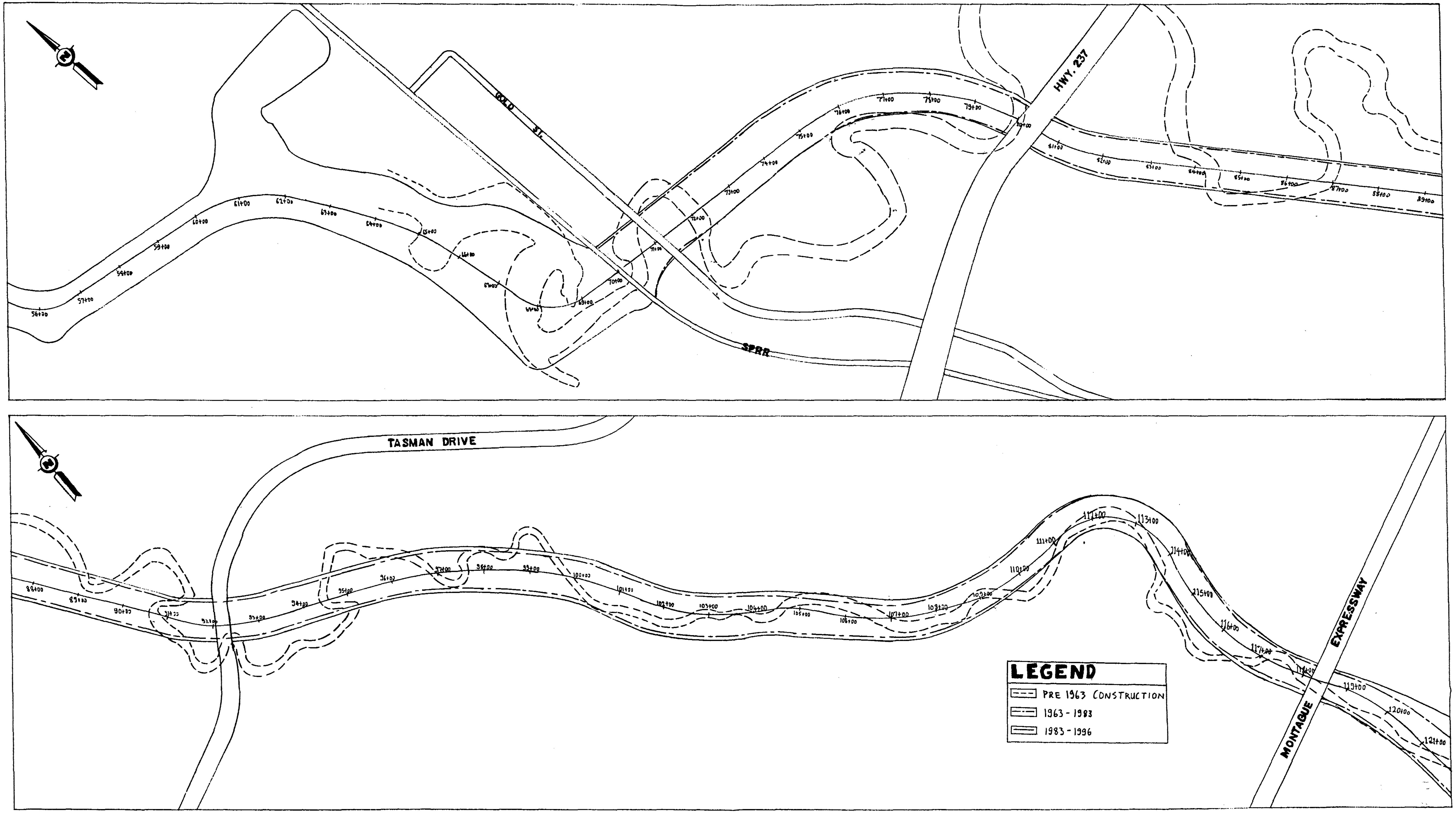
   Option-3: Acquire land to extend the District's right of way beyond the toe of the outboard slopes of both levees. Strip off the existing levees, realign if needed, and rebuild to the desired height with flatter slopes from the base as required.

It should be noted that an additional geotechnical investigation will be required to prepare a final design following any of the options outlined above.

As the existing levees continue to settle, develop cracks and shallow slides, Options 1 and 2 will require extensive maintenance services throughout the life of the project, and Option 3 will provide a better quality design and construction of new levees, thereby reducing the cost of maintenance services for the project. Therefore, it is recommended that the economic feasibility of different options as outlined above be evaluated for new levee design and a final project plan be prepared as soon as possible to proceed with the plan for additional geotechnical investigation to meet the project schedule.

APPENDIX-A

Site Plan (Plan View of Levee Alignment) and Levee Cross Sections



PLAN VIEW OF
TOP OF LEVEE OR
CHANNEL BANK

Note: Stationing is In meters.
Scale is 1:3600.

DATE
6/1/96

DESIGN
T. BERTULIS

DRAWN
T. BERTULIS

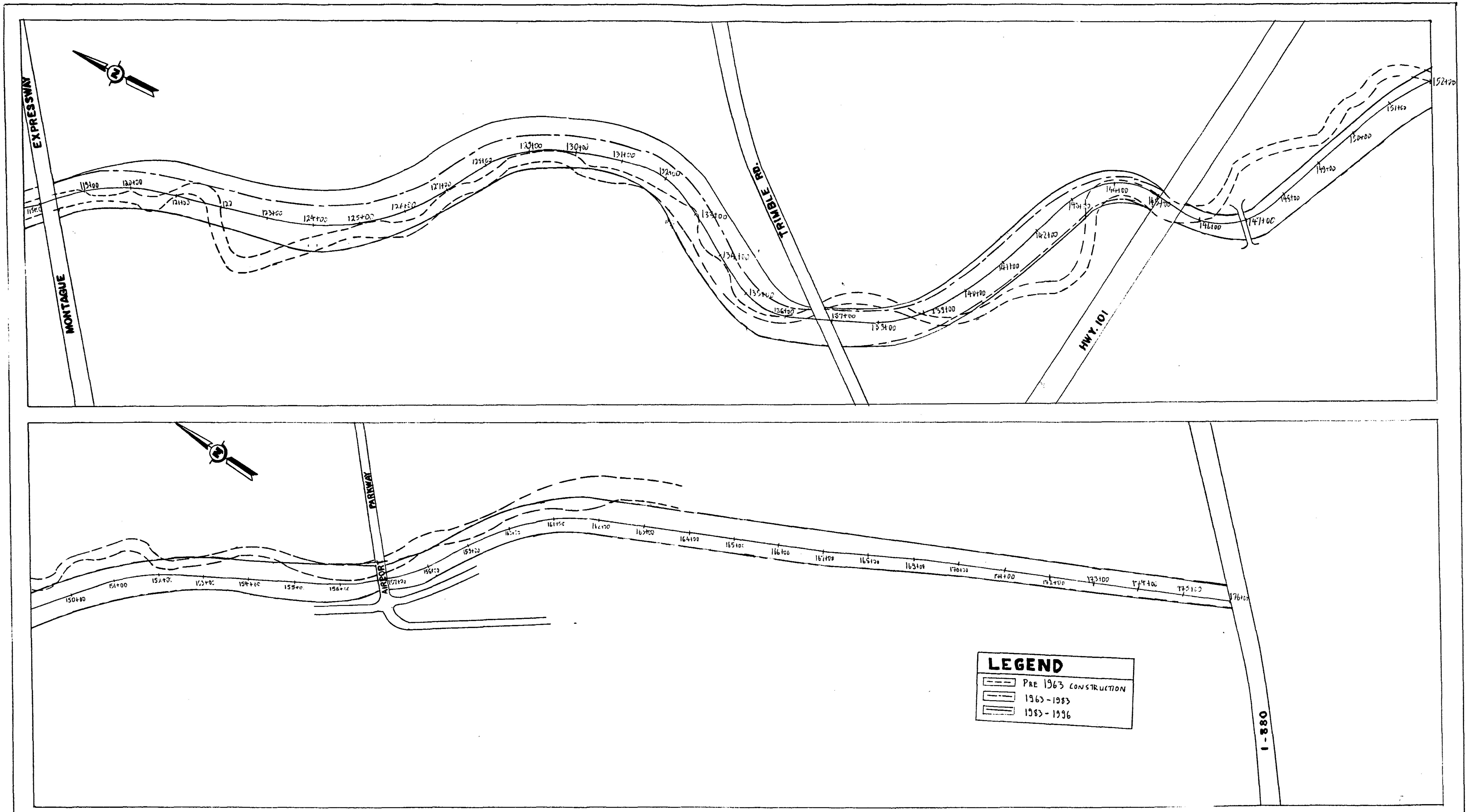
CHECKED
P. SHEYDAYI

LOWER GUADALUPE RIVER PROJECT

ALVISO TO I 880

SCALE
1" = 300'±

SHEET
1 OF 2



PLAN VIEW OF
TOP OF LEVEE OR
CHANNEL BANK

Note: Stationing is in meters.
Scale is 1:3600.

DATE
5/1/96
DESIGN
T. BERTULIS
DRAWN
T. BERTULIS
CHECKED
R. SHEYDAYI

LOWER GUADALUPE RIVER PROJECT

ALVISO TO I 880

SCALE
1" = 300'±
SHEET
2 OF 2

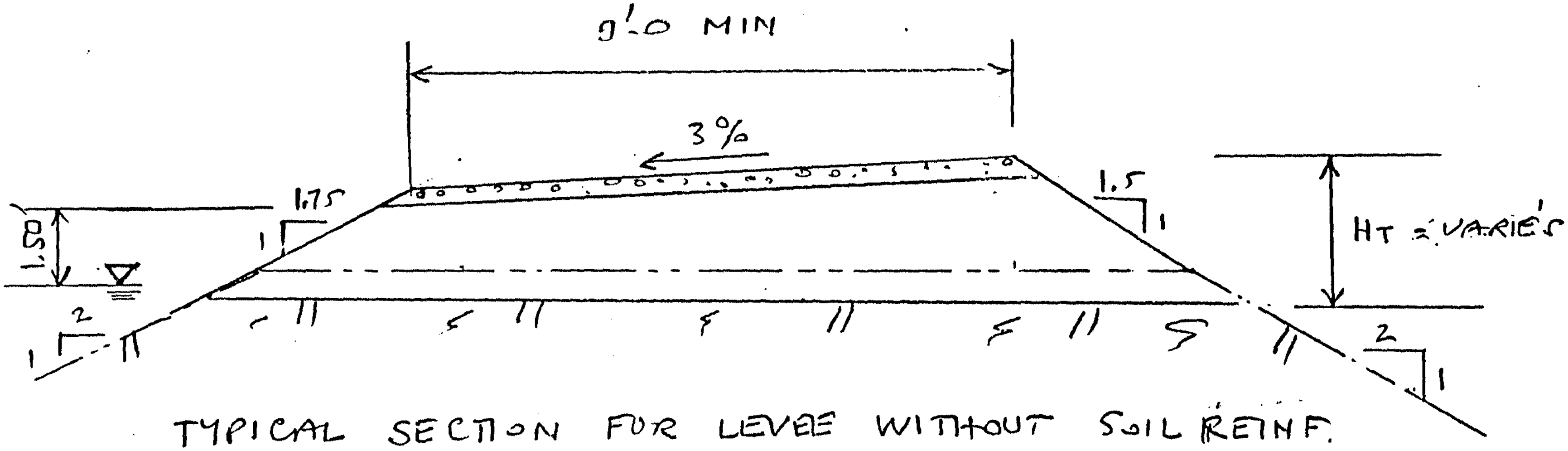
5750 Almaden Expressway, San Jose 95118, Phone 265-2600

Subject GUADALUPE RIVER

Date 8-22-1995

Checked by ______ Date ______

TYPE 1



TYPICAL SECTION FOR LEVEE WITHOUT SOIL REINF.

SCALE : 1" = 30'-0

(EXCEPT TYPE 1B AT WEST BANK FROM STA 45+70 TO 52+50)

MAIN REQUIREMENTS:

1- MIN. LEVEE WIDTH = 9'-0"

2- NEW INBOUND SLOPE = 1.75:1

3- NEW OUTBOUND SLOPE = 1.50:1

FC 807 (10-21-92)

printed on recycled paper

Subject GUADALUPE RIVER

Page 3 of

Charge No.

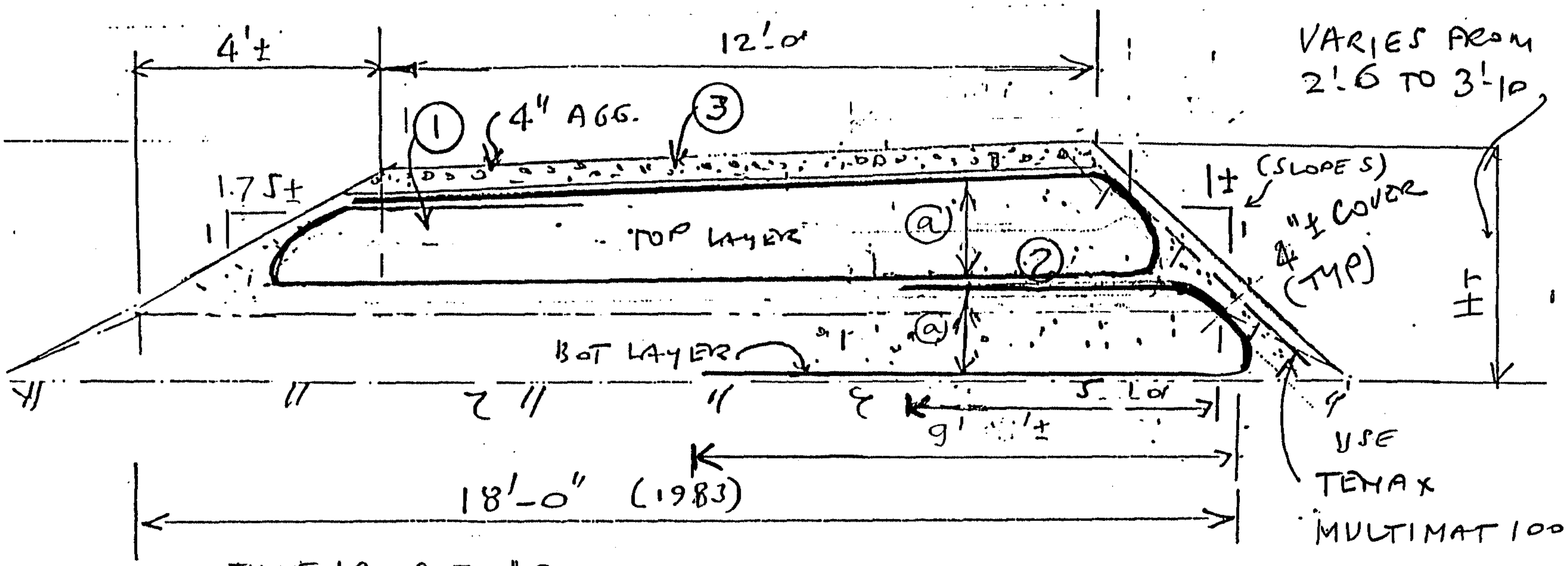
By TRI NGUYEN

Date

Checked by ______ Date ______

PRELIM

TYPE 1A-



TYPE-1A ALT. #2

TABLE 1-

HT	SLOPE S	TOP LAYER	BOTTOM LAYER	@ SPACING	REMARK
2'-6"	1 1/4:1	YES	NONE	24"	
3'-0"	1:1	YES	YES	14"±	USE TENAX 100
3'-10"	3/4:1	YES	YES	21"	

FC 807 (10-21-92)

Subject GUADALUPE - LEVEE - REACH - #1

Page 2 of

Charge No.

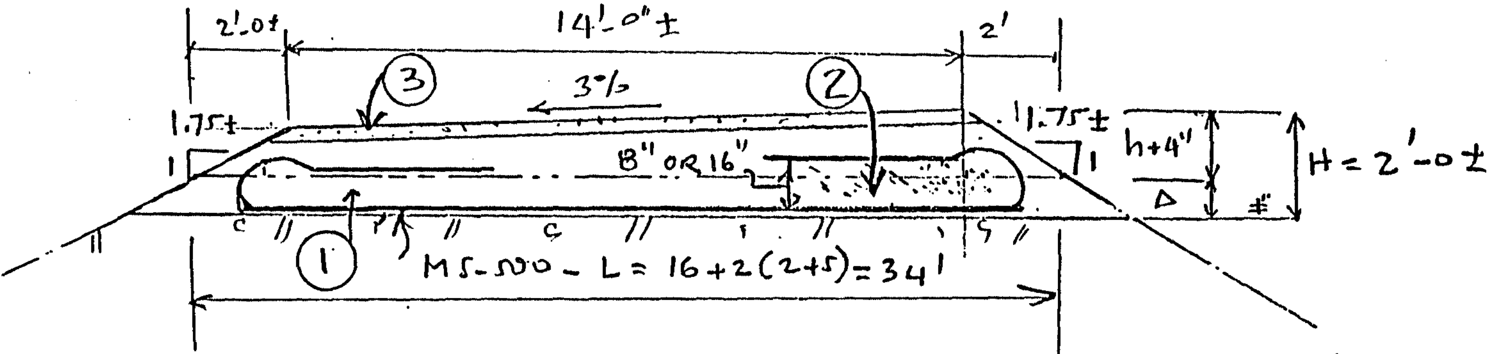
By TRI NGUYEN

Date 8-2-1995

Checked by Date

LOAD : 1 TRUCK HS-20- OR
EQUIVALENT 2'-0 SURCHARGE

MATERIALS FILL:
① IMPERVIOUS SANDY CLAY, P.I ≤ 25 ± 5
② SEMI-IMPERVIOUS, OR STRUCTURAL BACKFILL, P.I ≤ 15 ± 5
③ 4" AGGREGATE BASE SURFACING



TYPE-1B- WEST BANK - STA 45+70 TO 52+50
(REINF. TO MINIMIZE TENSION CRACK. ON SOFT BAY MUD CLAY)

TYPE 1B.

Santa Clara Valley Water District
5750 Almaden Expressway, San Jose 95118, Phone 265-2600

Subject GUADALUPE RIVER

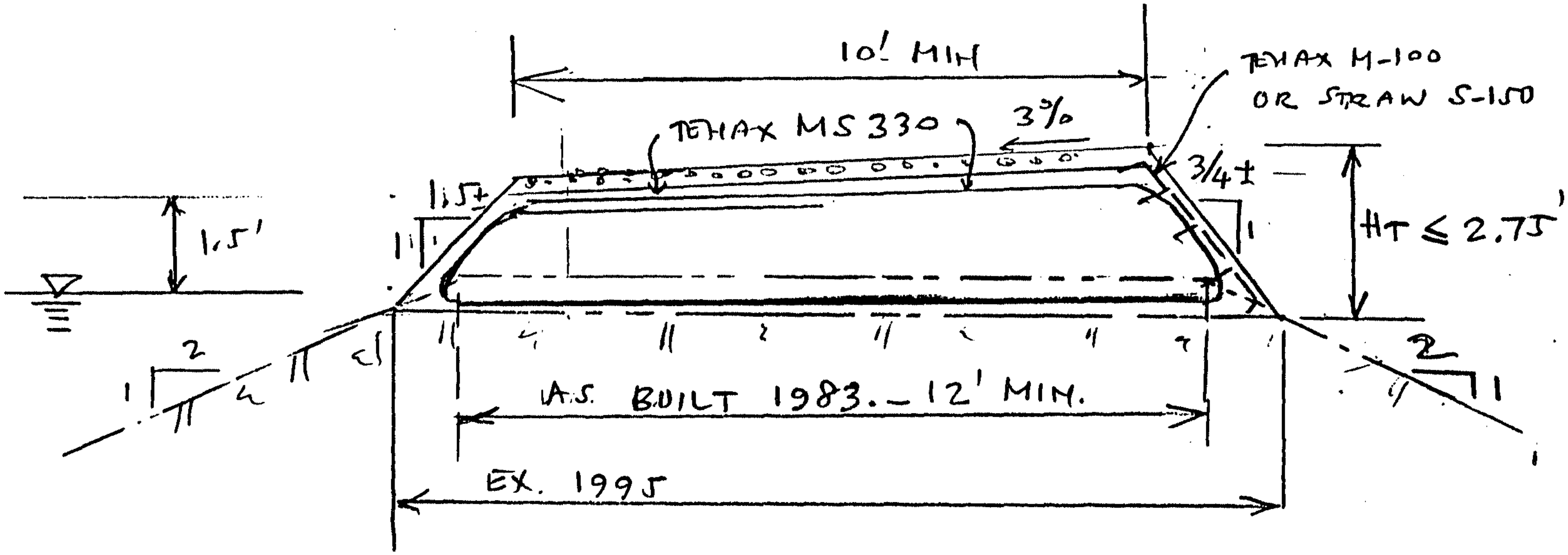
Page 4 of

Charge No.

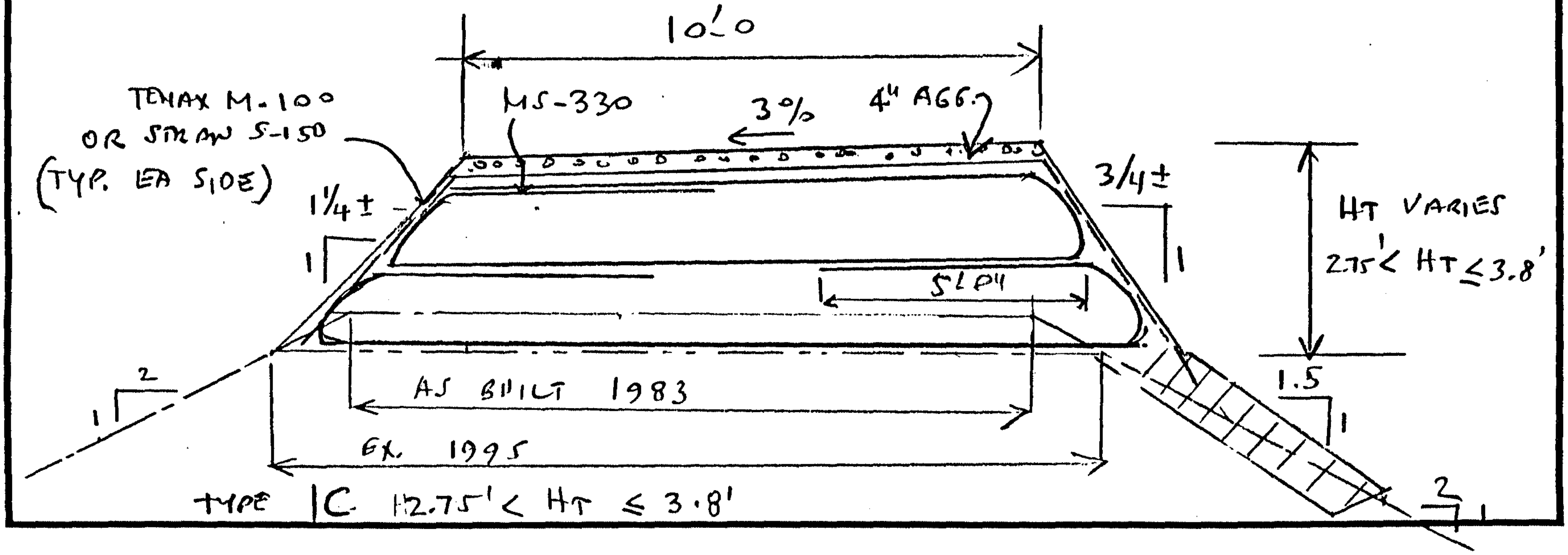
By TRI NGUYEN

Date 8-22-1995

Checked by ______ Date ______



TYPE 1C. HT ≤ 2.75' - SCALE 1" = 30'



TYPE 1C H2.75' < HT ≤ 3.8'

printed on recycled paper

Santa Clara Valley Water District
5750 Almaden Expressway, San Jose 95118, Phone 265-2600

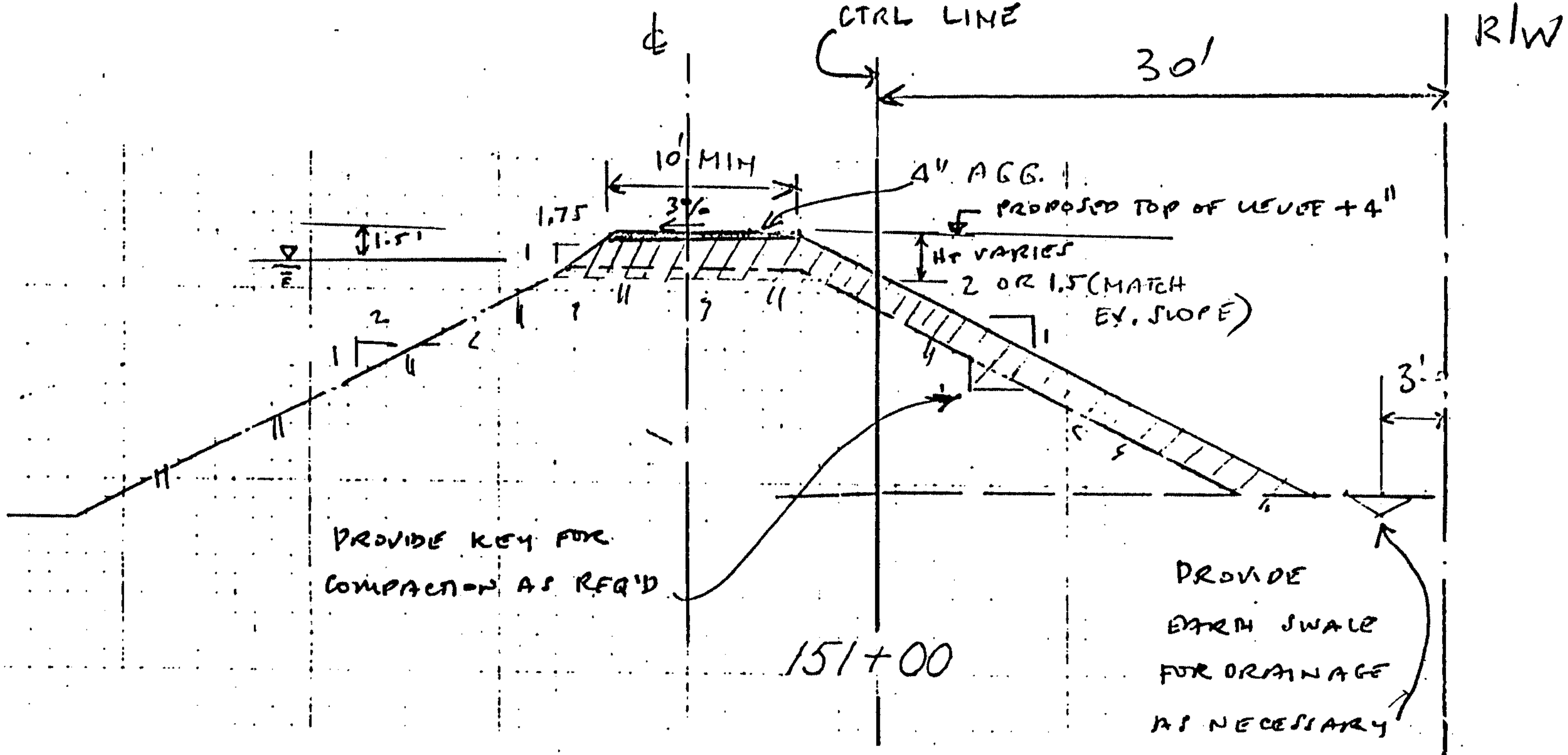
Subject GUADALUPE

Charge No.

By DRC NGUYEN

Date 8-23-1995

Checked by ______ Date ______



℄
CTRL LINE
R/W
30'
10' MIN
4" AGG.
3%
1.75
1
1.5'
PROPOSED TOP OF LEVEE + 4"
Ht VARIES
2 OR 1.5 (MATCH EX. SLOPE)
2
1
2
1
1
3'
PROVIDE KEY FOR COMPACTION AS REQ'D
151+00
PROVIDE EARTH SWALE FOR DRAINAGE AS NECESSARY

TYPICAL OVERLAY LEVEE - 1 (WITHIN R/W)

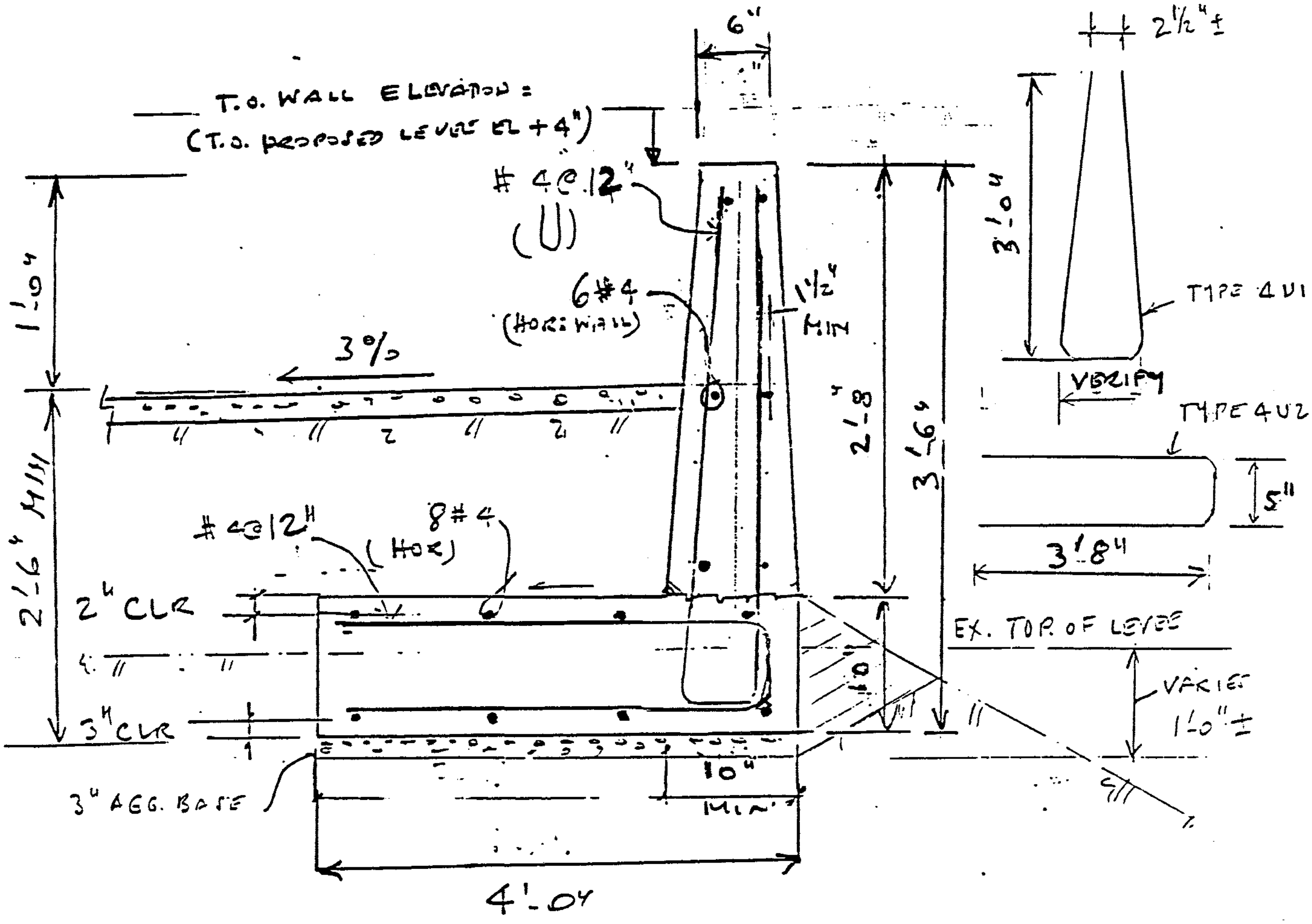
FC 807 (10-21-92)

printed on recycled paper

Santa Clara Valley Water District
5750 Almaden Expressway, San Jose 95118, Phone 265-2600

Subject GUADALUPE RIVER.

By TRI NGUYEN
Date 8-22-1995
Checked by ______ Date ______

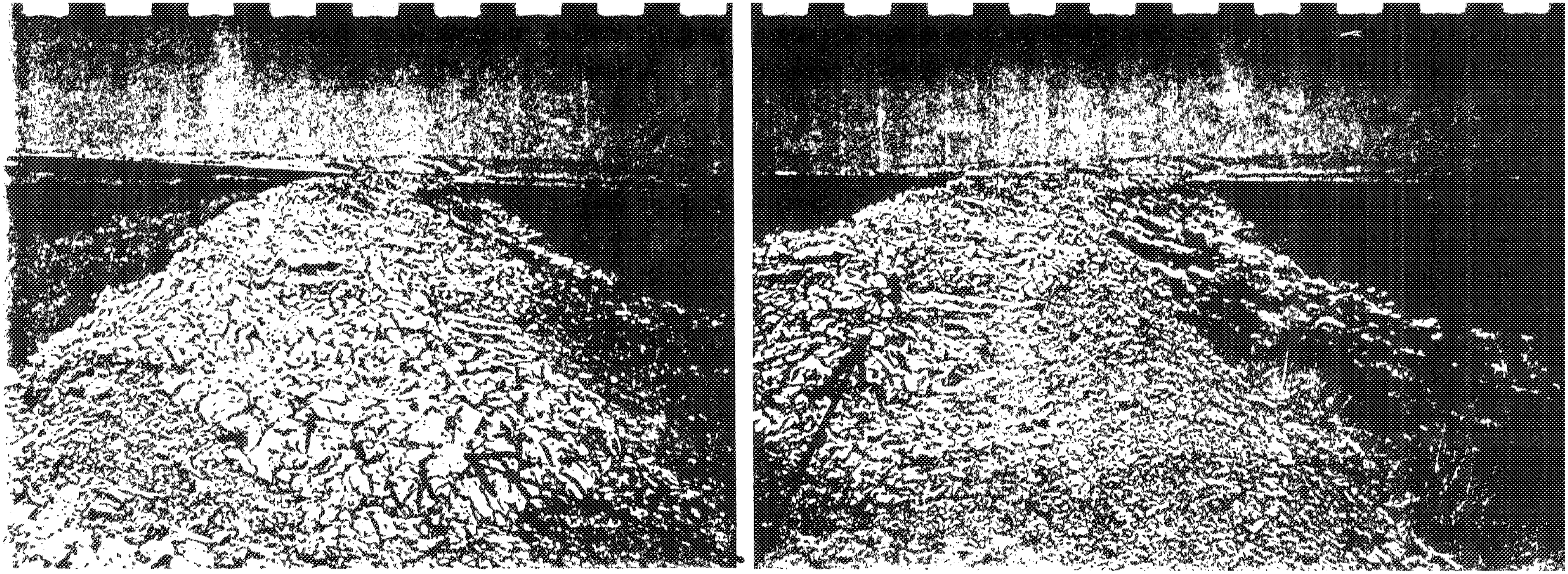


TYPE 2

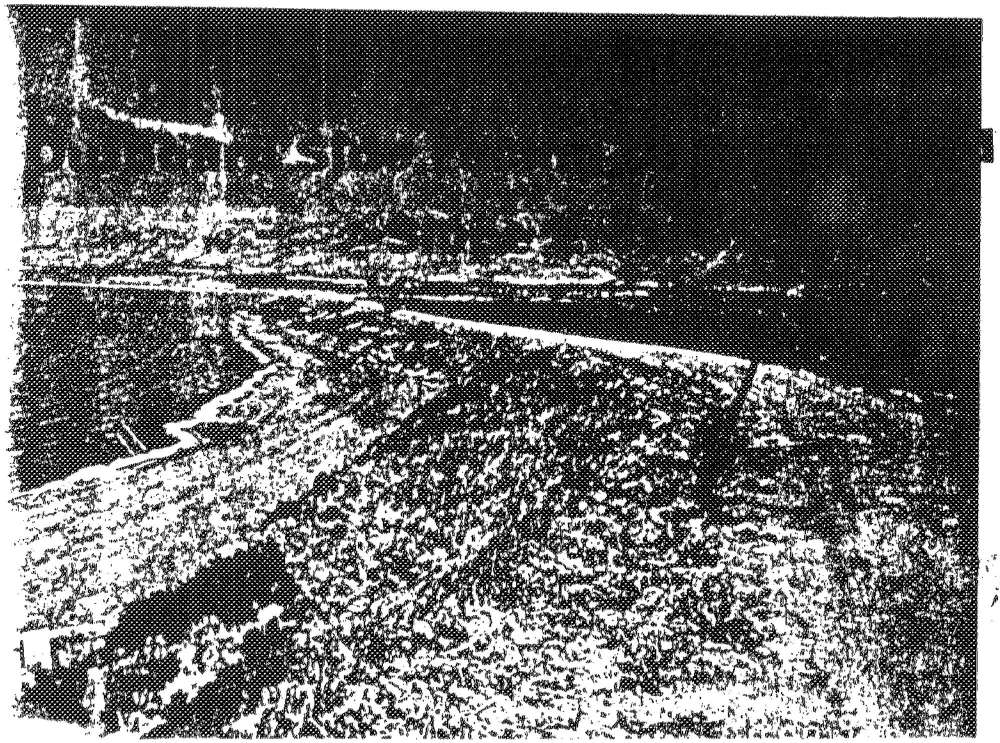
printed on recycled paper

APPENDIX-B

Photographs of Significant Surface Features

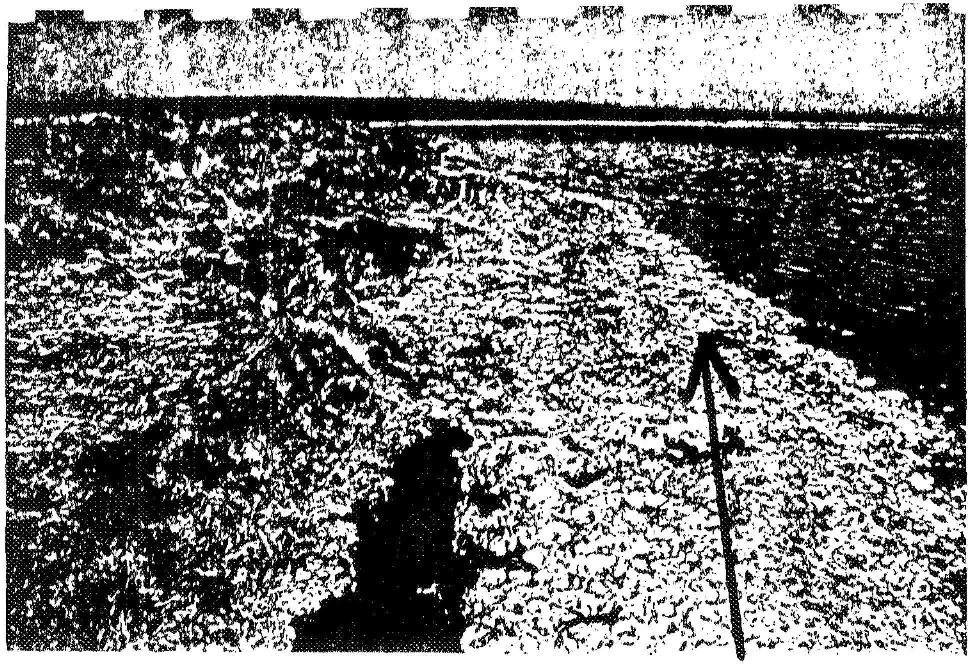


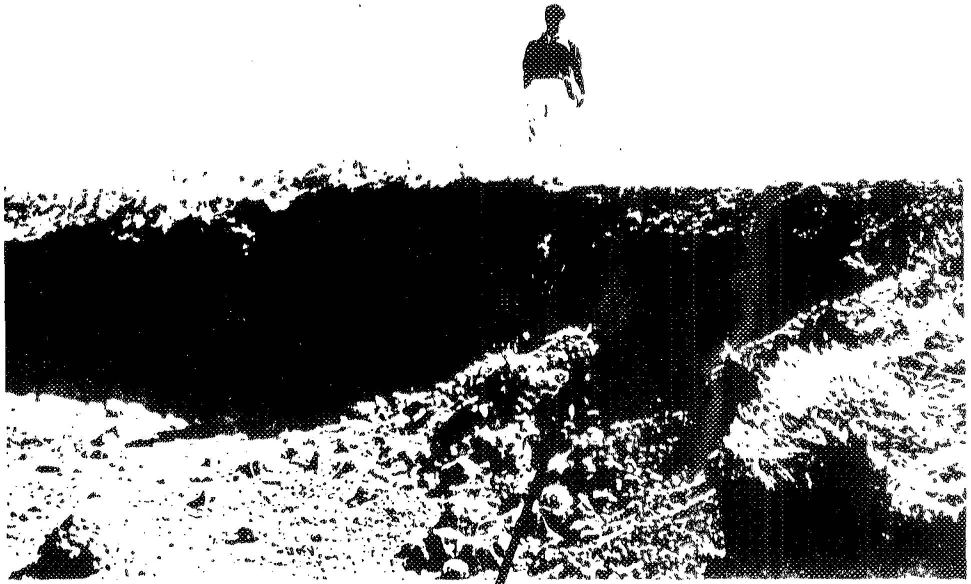
East Levee @ Station 53+00, Dredge Sediment



t Levee @ Station 59+00

Uneven Levee Settlement

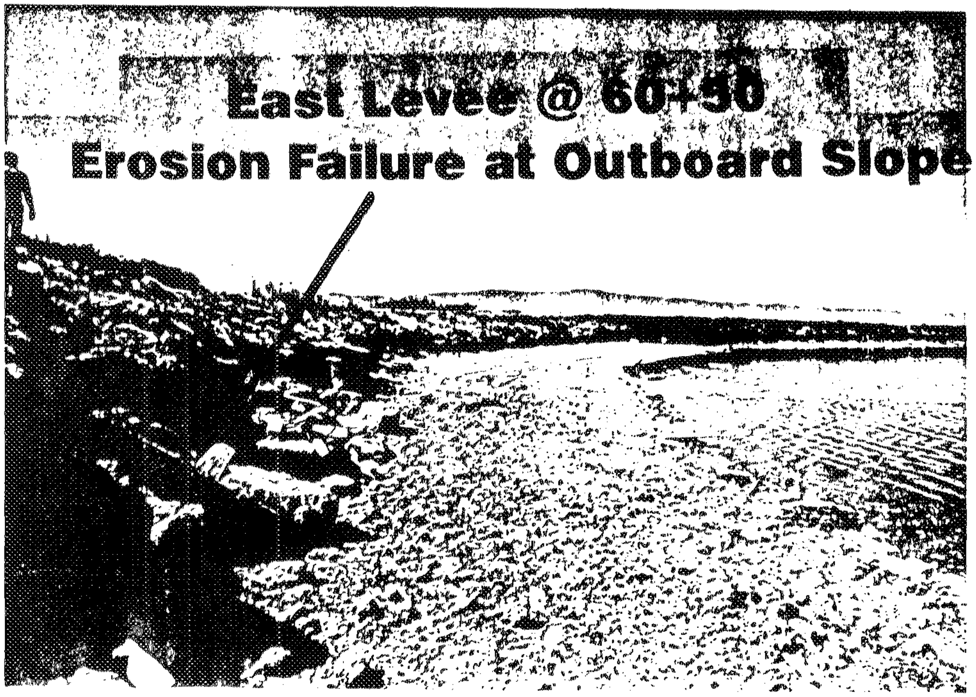




East Levee @ Station 60+00

Outboard Slope Facing Aeration Pond

-Slope Failure



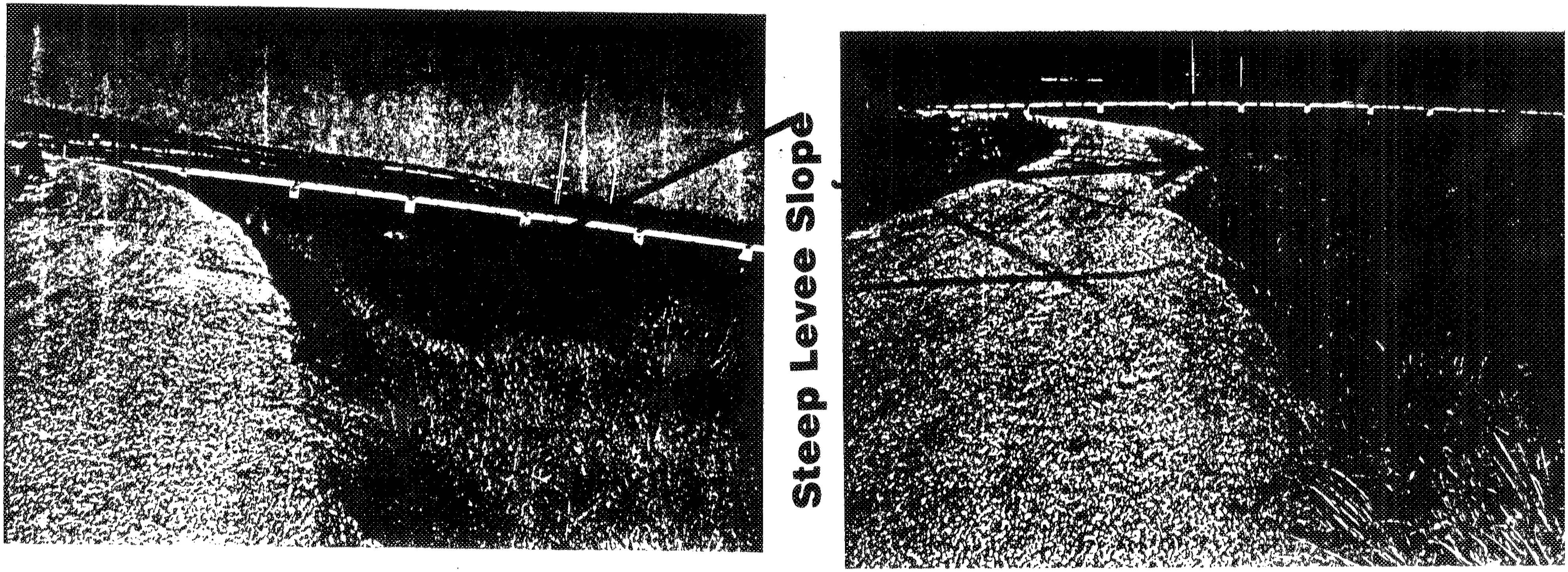
East Levee @ 60+50

Erosion Failure at Outboard Slope

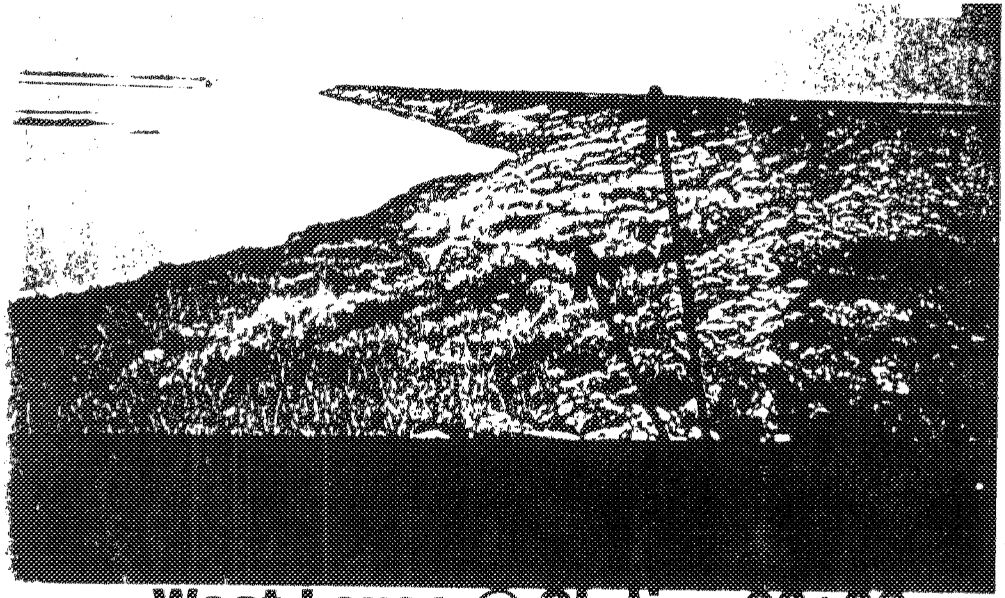
West Levee @ Station 67+00

Concrete Rubble

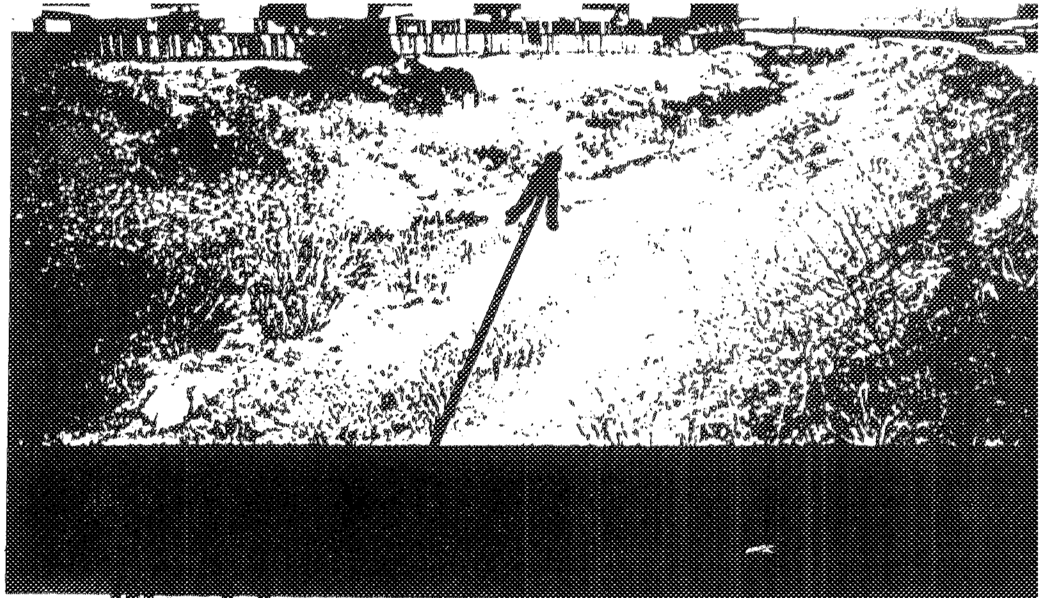
Blocking Levee Entrance



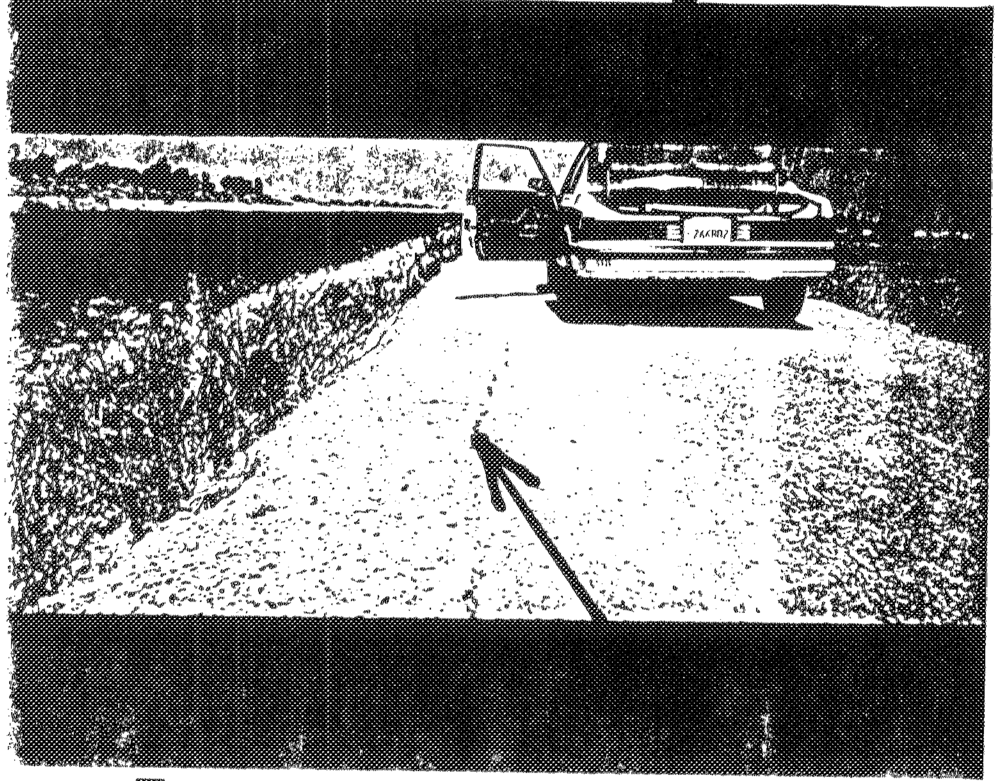
West Levee between SPRR & Gold St.
(Station 70+00 to 71+00)



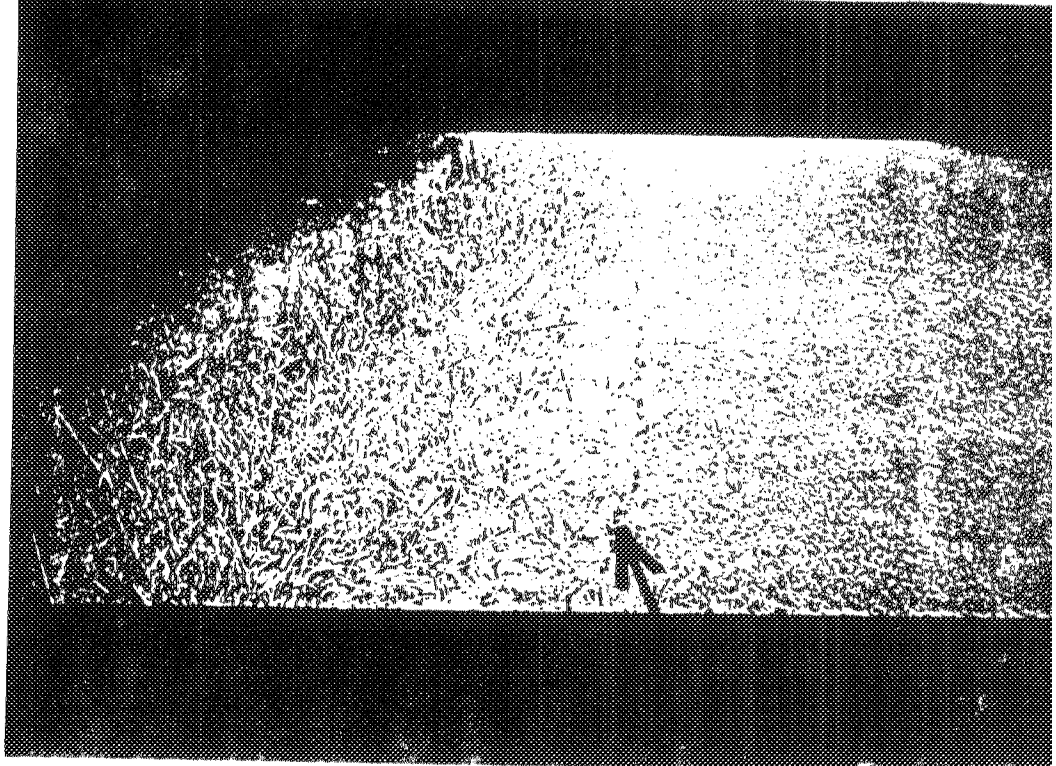
West Levee @ Station 68+00
Outboard Side Facing Salt Pond



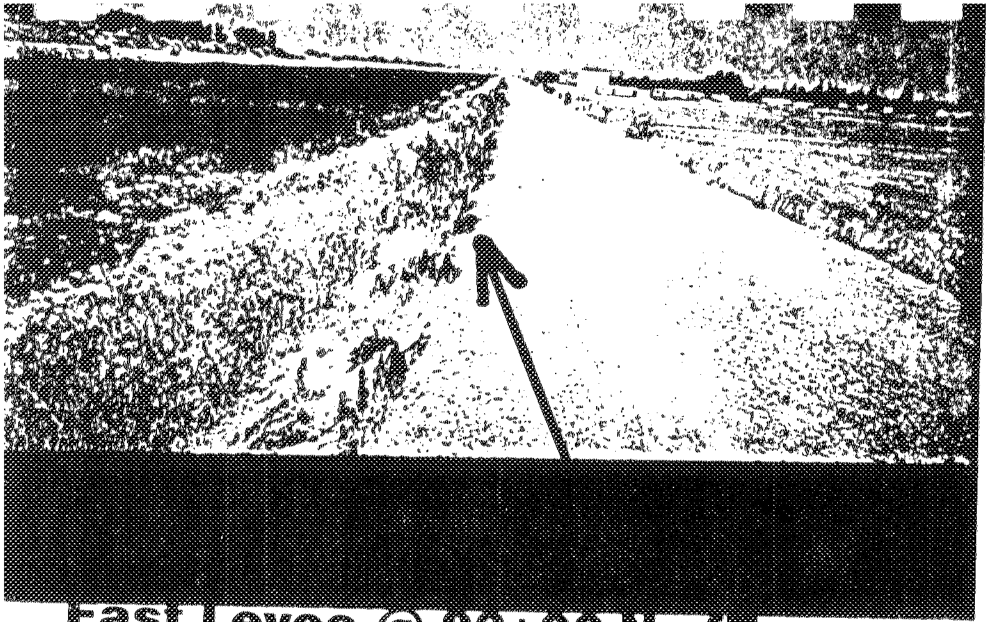
West Levee @ Station 77+00
Deep Ditch at Outboard Side



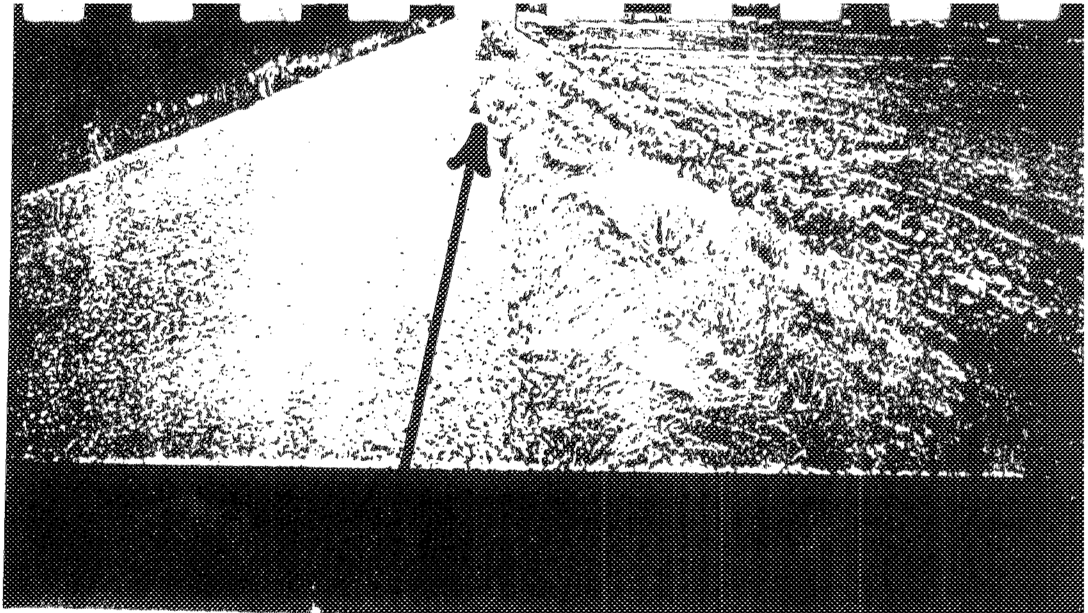
East Levee @ Station 78+00
Crack @ 2 feet from inboard side



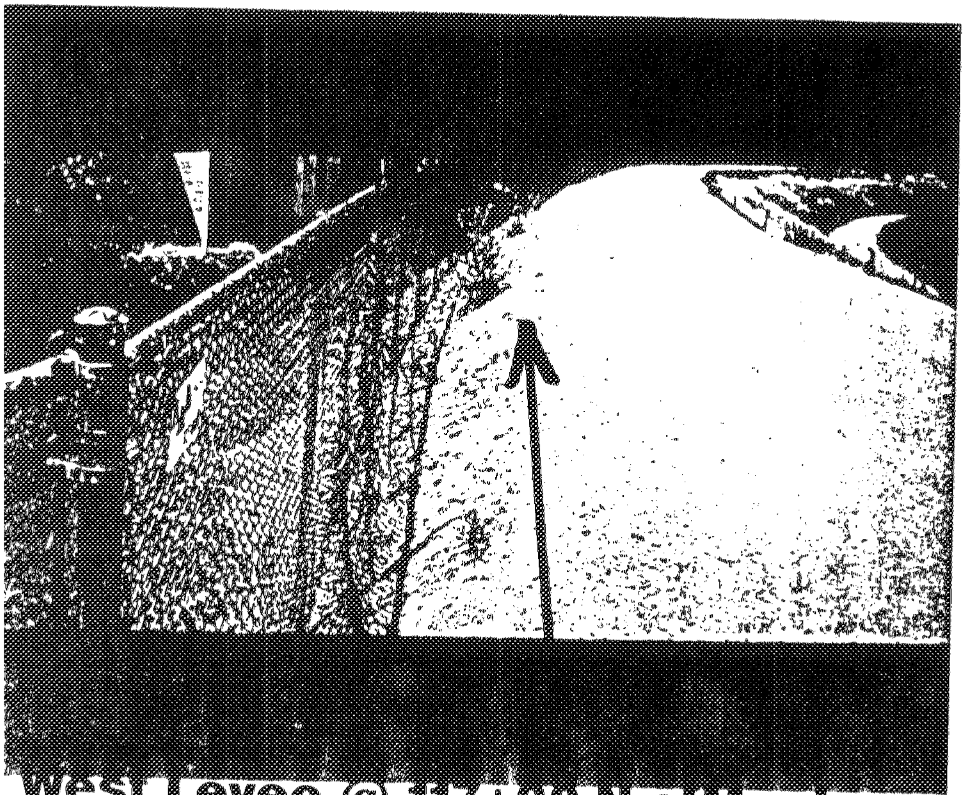
East Levee @ Station 78+00
Crack at outboard side



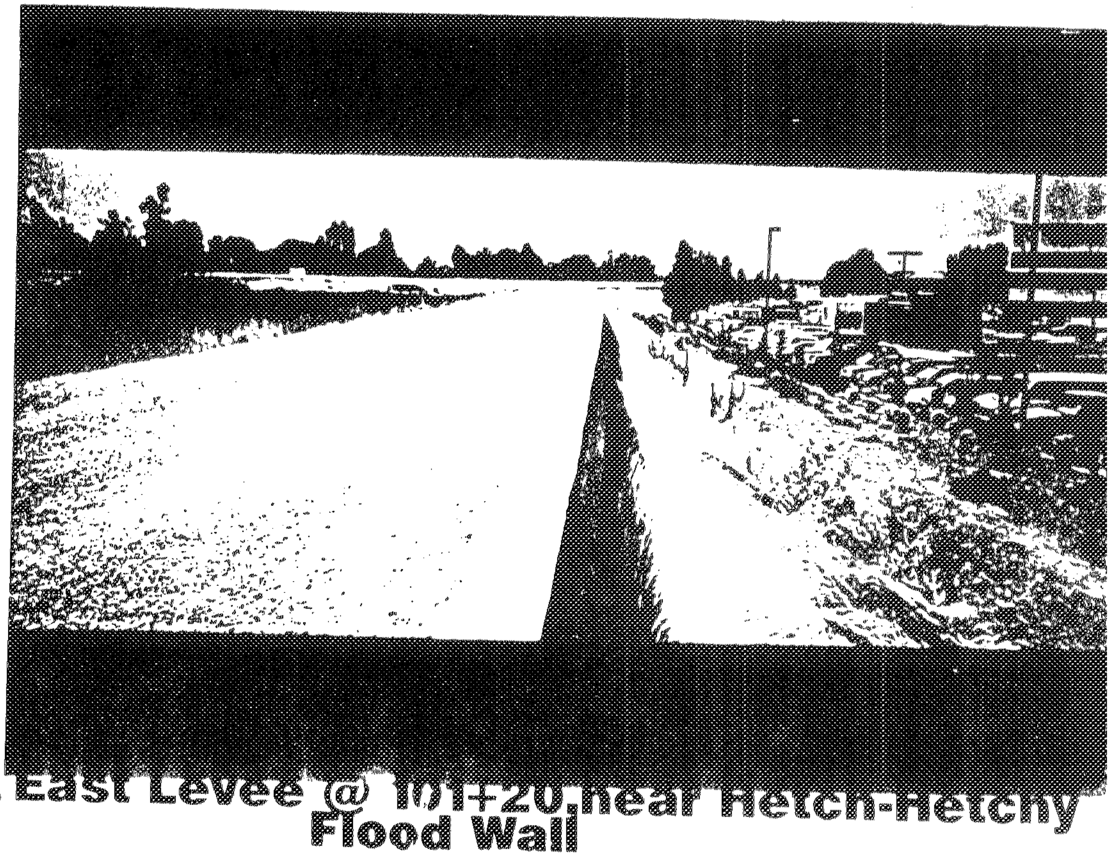
East Levee @ 92+00,N.of Tasman
Steep inboard slope



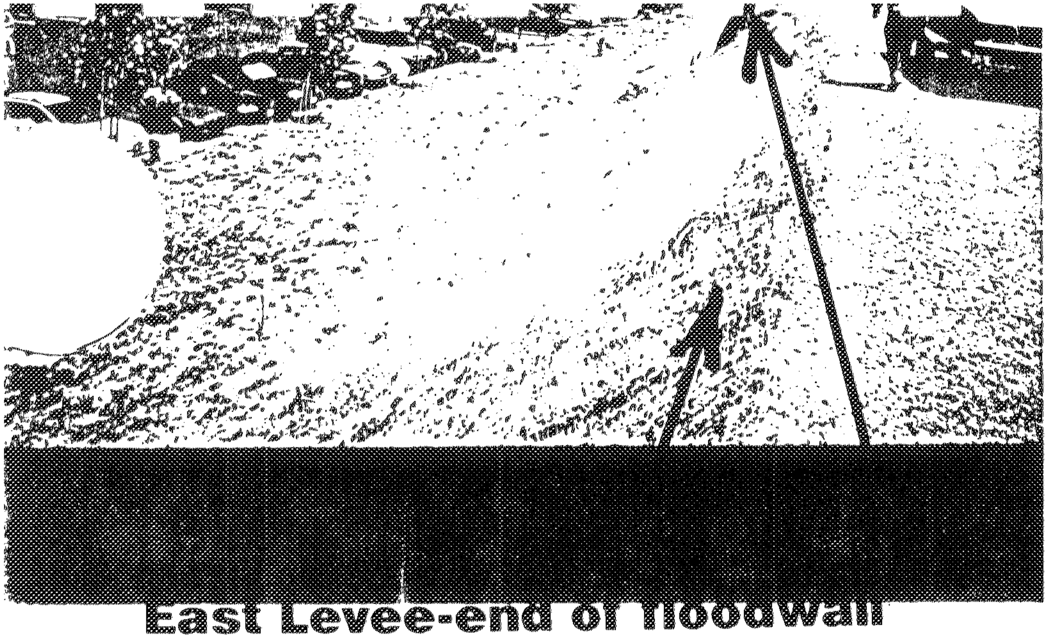
East Levee @ 92+00
Steep outboard slope



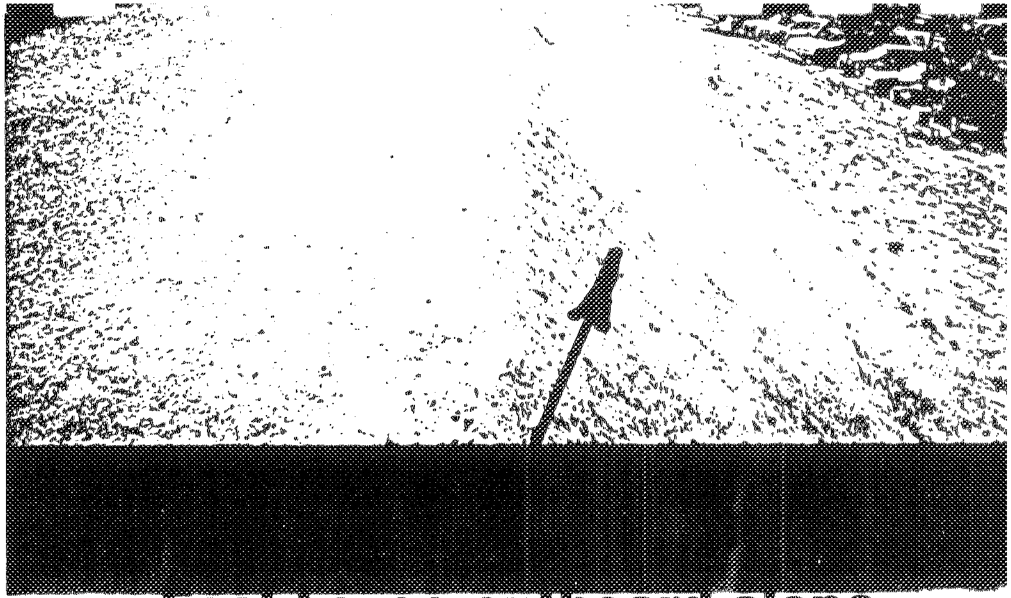
West Levee @ 117+00,N.ofMontague
Steep slope



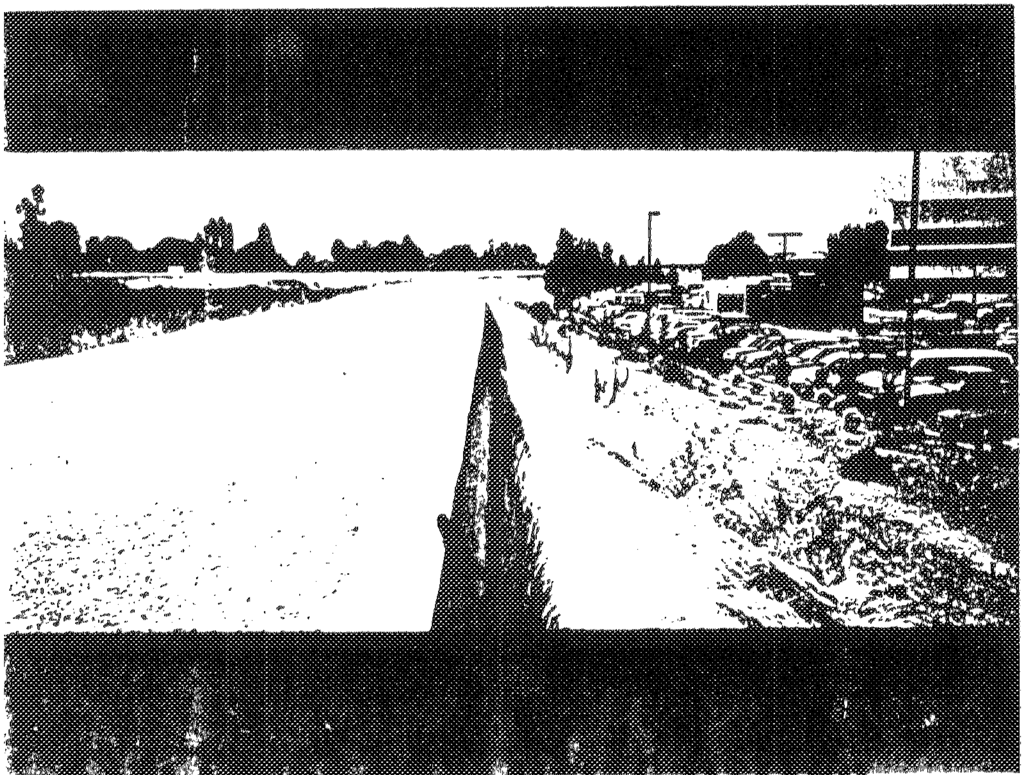
East Levee @ 101+20,near Hetch-Hetchy
Flood Wall



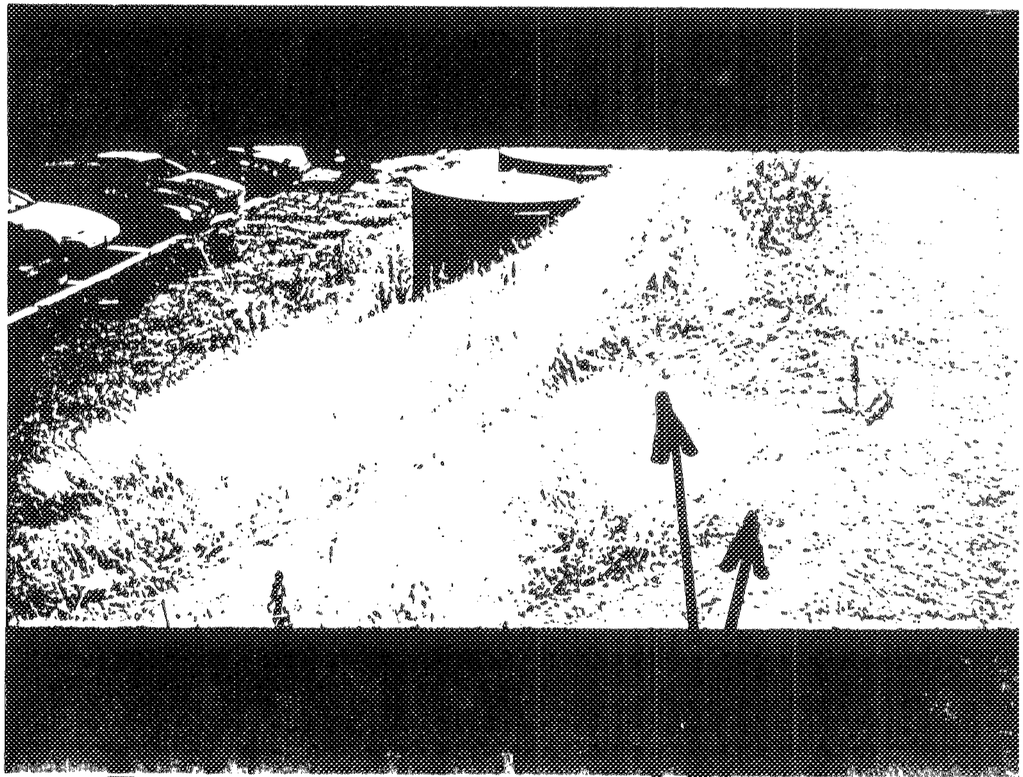
East Levee-end of floodwall
Broken Slope Protection Mat @ 96+60



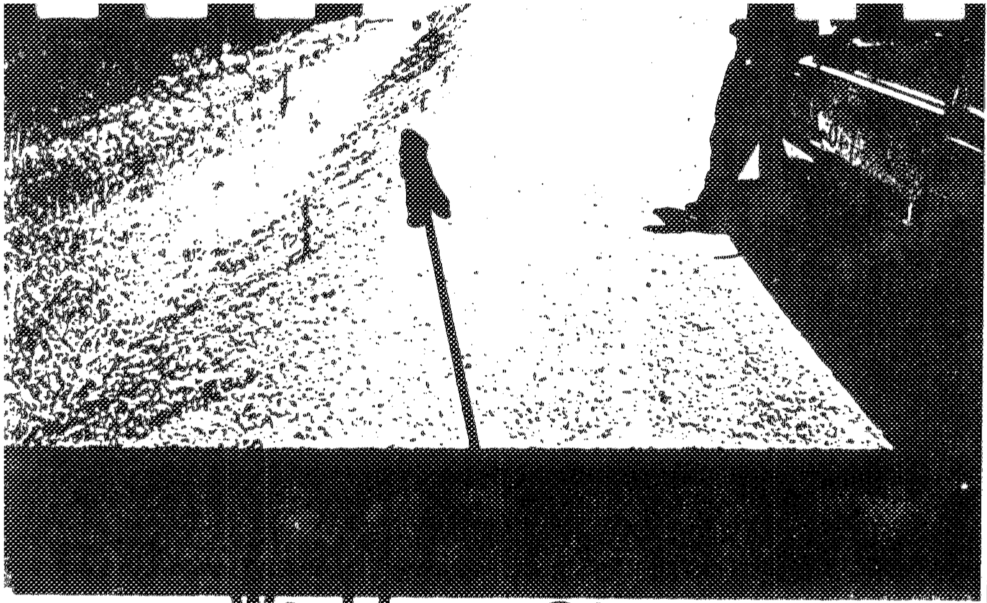
East Levee-outboard slope
Broken Slope Protection Mat @ 96+70



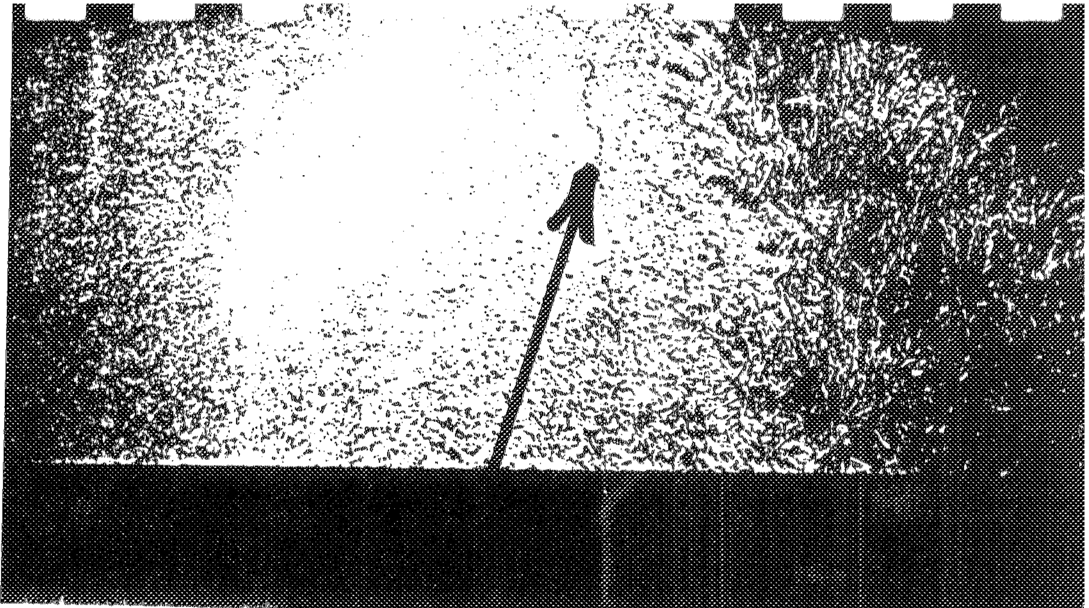
East Levee near Hetch-Hetchy
Floodwall @ Station 101+50



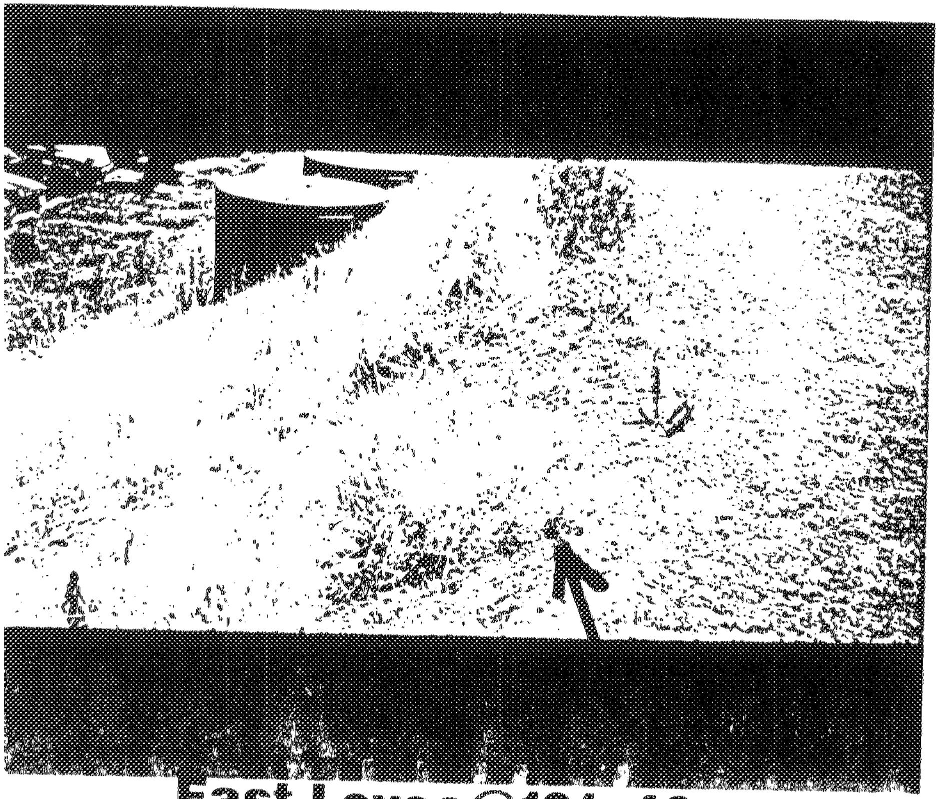
East Levee-Outboard Slope
Slope Erosion & Animal Burrow @ 101+70



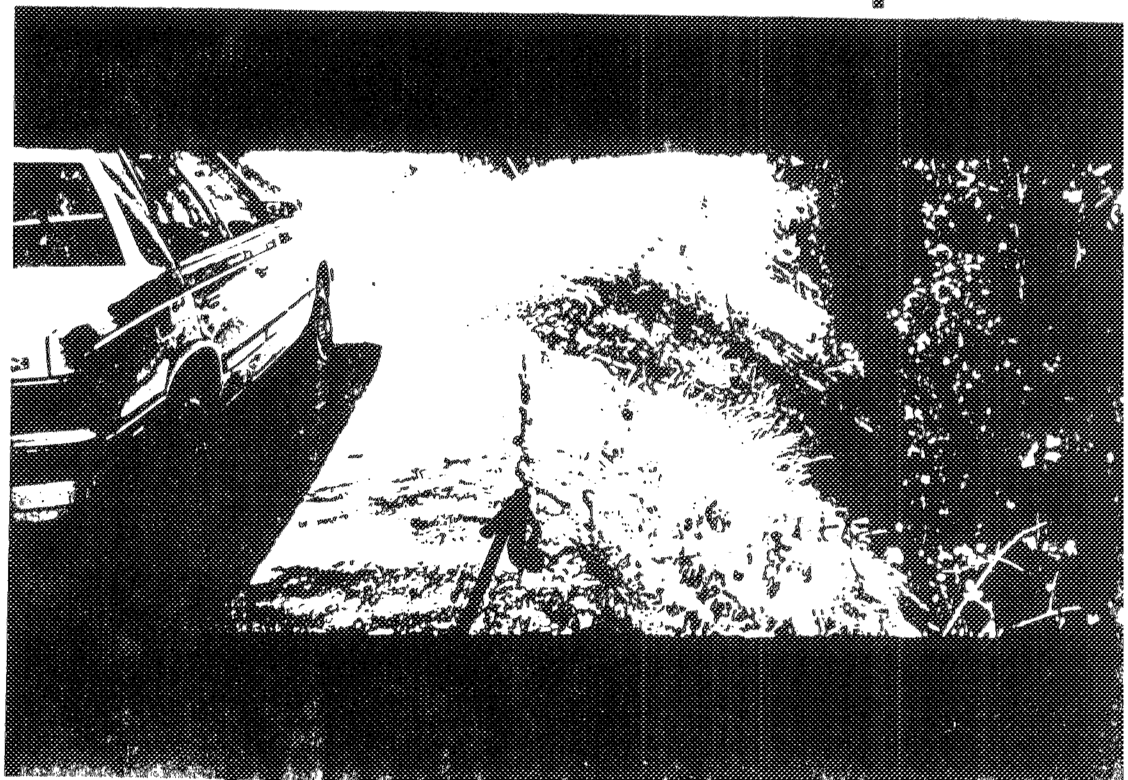
West Levee@93+00
Steep slope with crack



West Levee@107+00
Crack near inboard slope

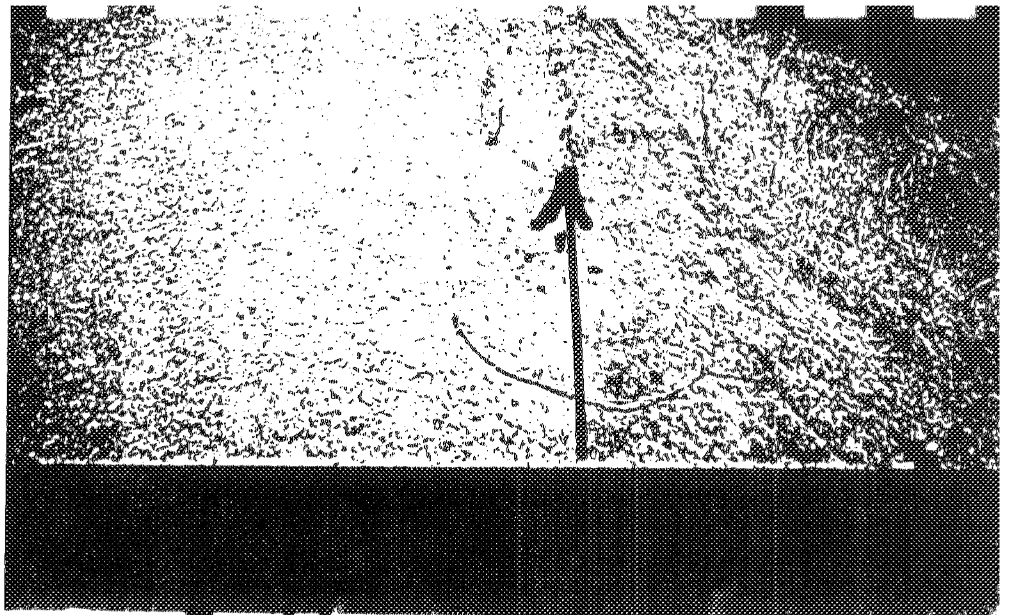


East Levee@101+40
Animal hole

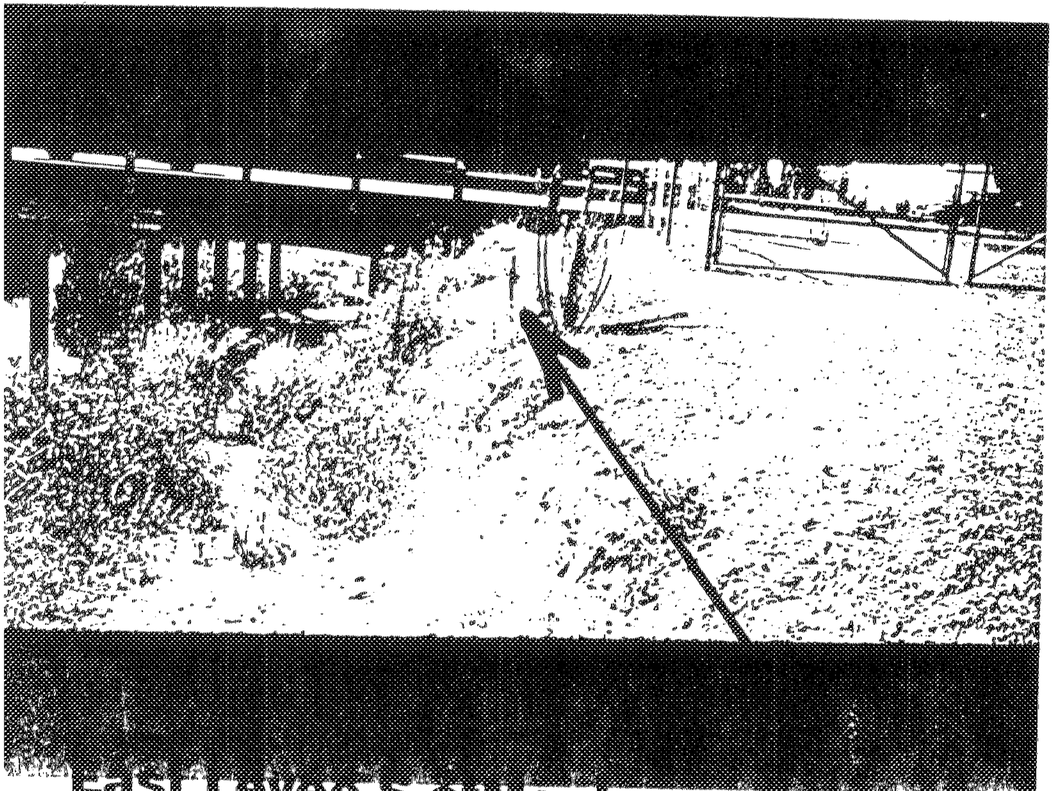


East Levee@118+00
Shallow slide,n.ofMontague

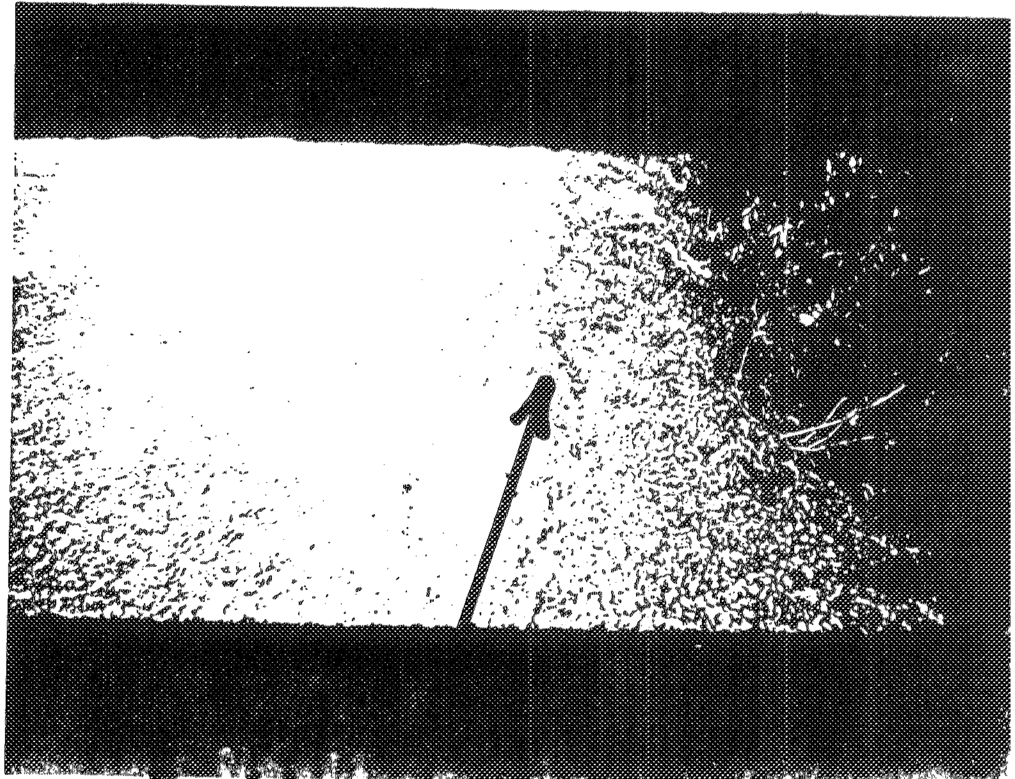
West Levee@117+00
Steep slope,N.ofMontague



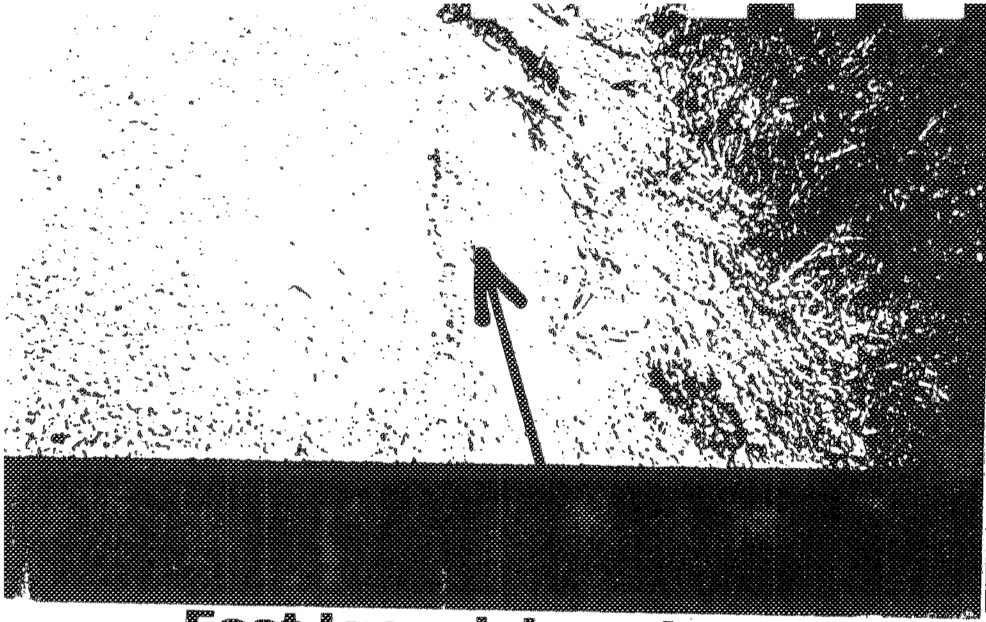
East Levee@119+00
Crack on open slope



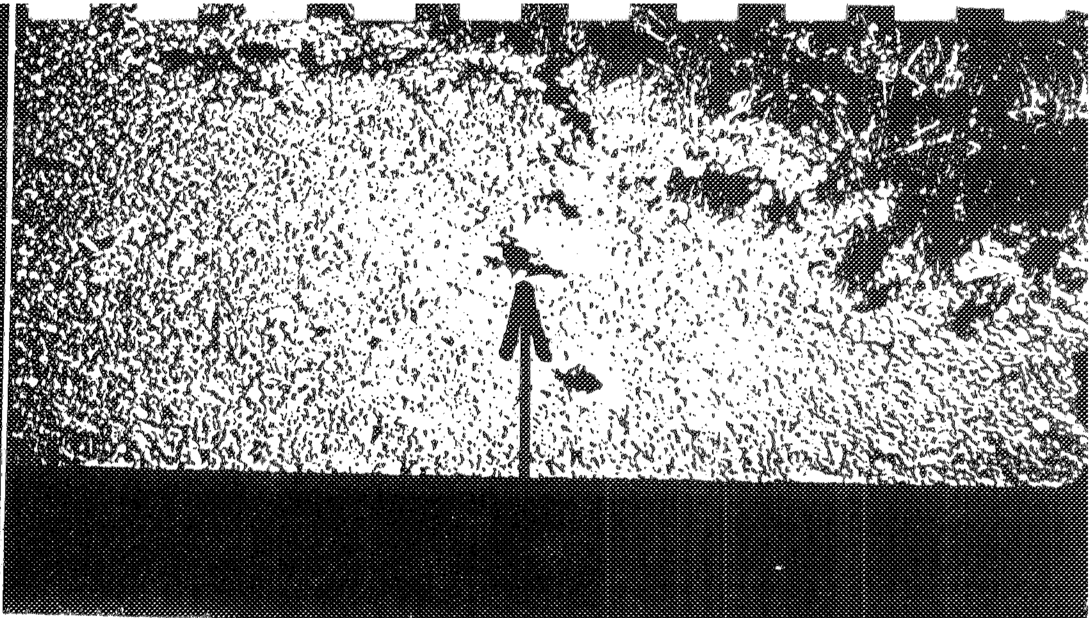
East Levee,S.ofMontague@118+40
Open slope @bridge abutment



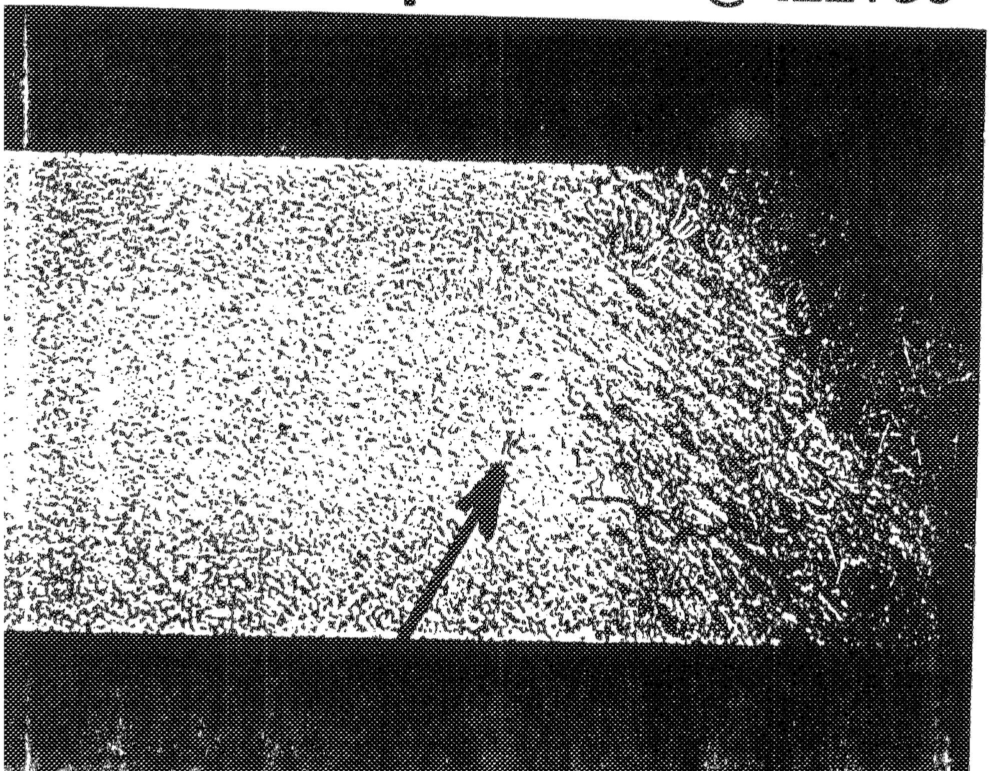
East Levee @118+00
Crack at outboard slope



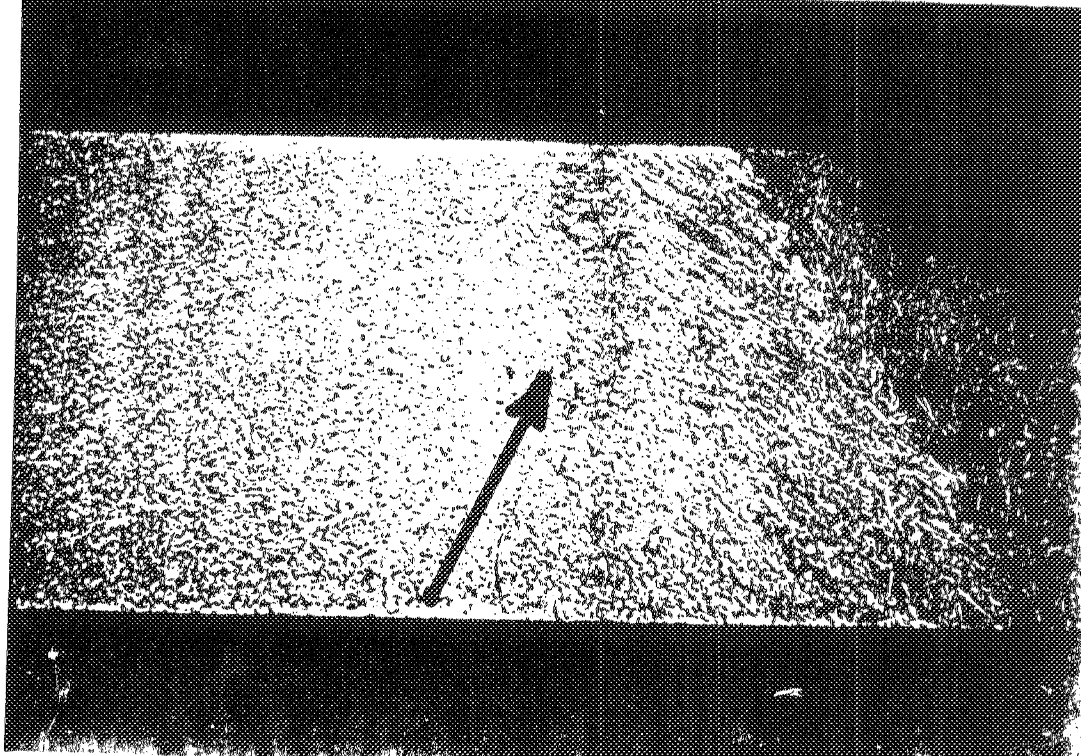
East Lvee -inboard side
Crack & Slope erosion @ 122+50



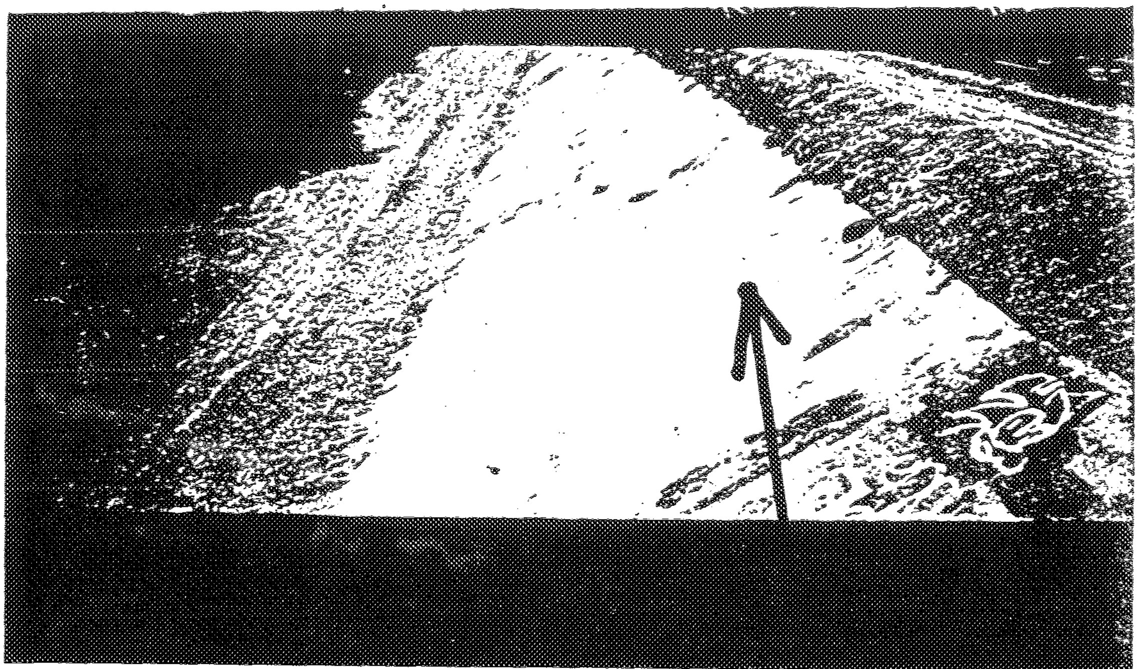
East Levee-inboard side
Animal burrow @ 121+00



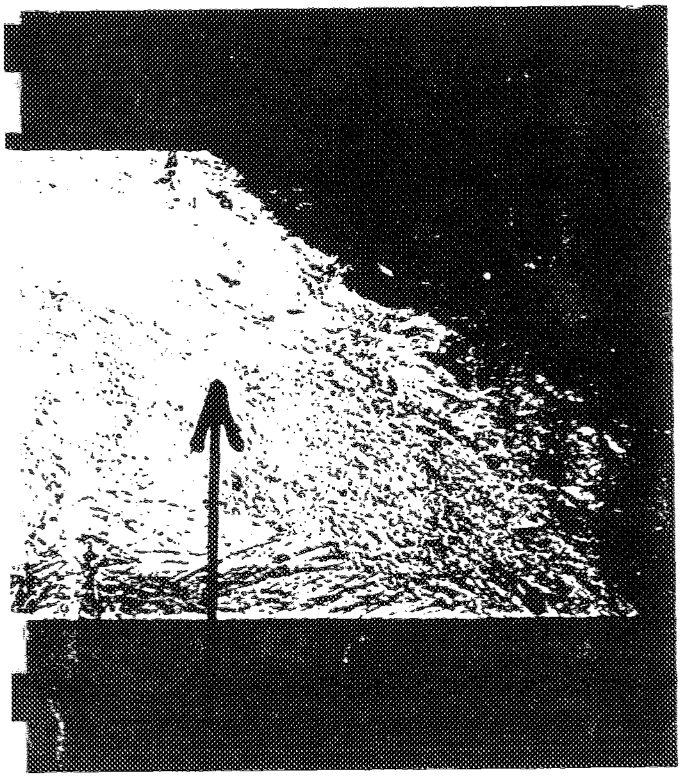
East Levee-outboard side
Crack @ 120+00



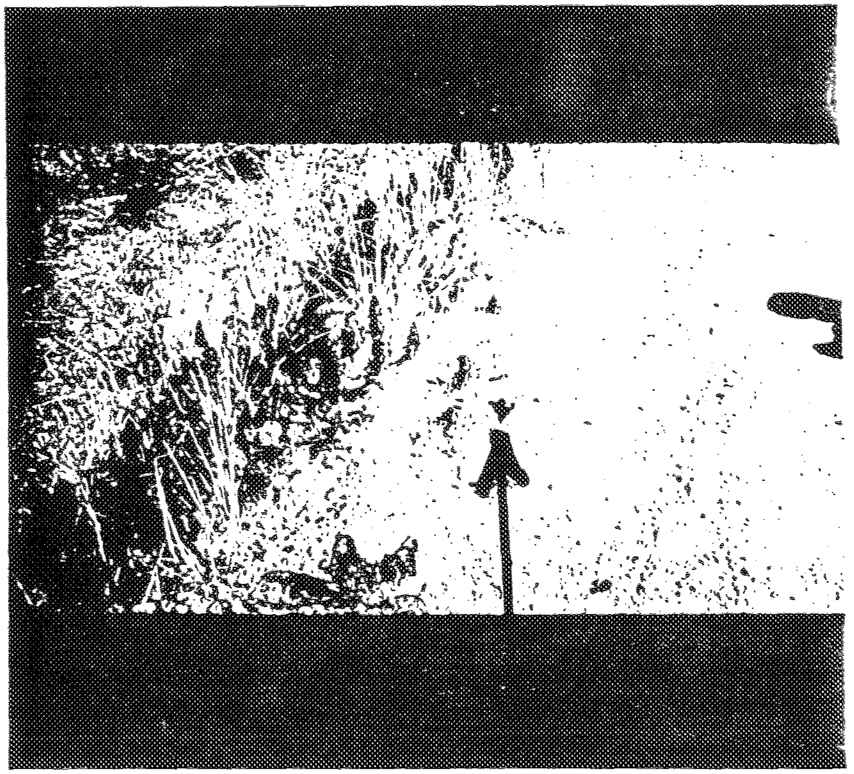
East Levee-inboard side
Crack @ 120+00



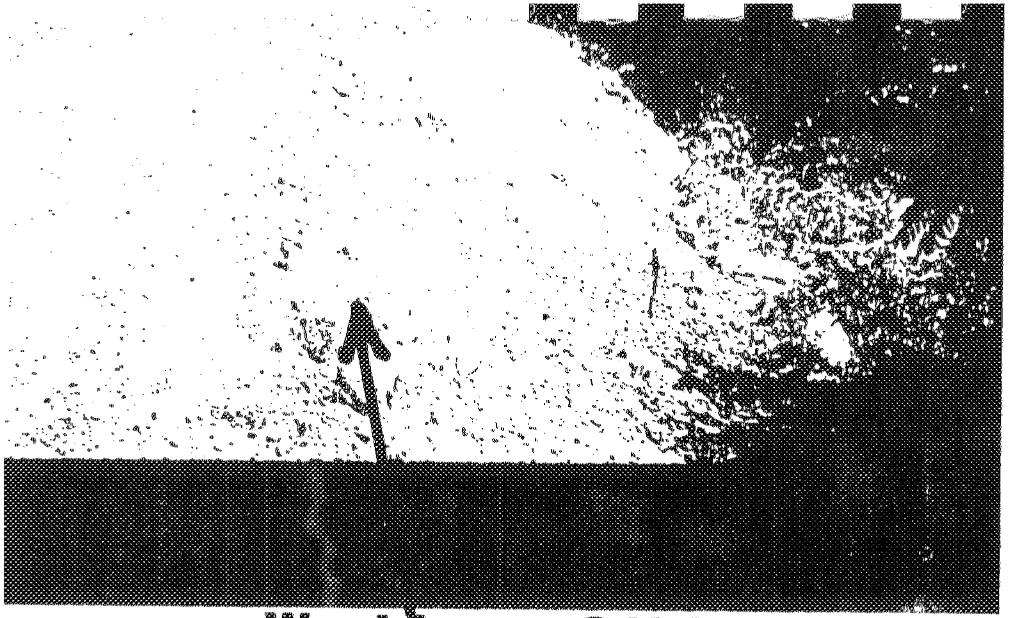
West Levee@118+50,S.of Montgaue
Concrete lining on inboard slope



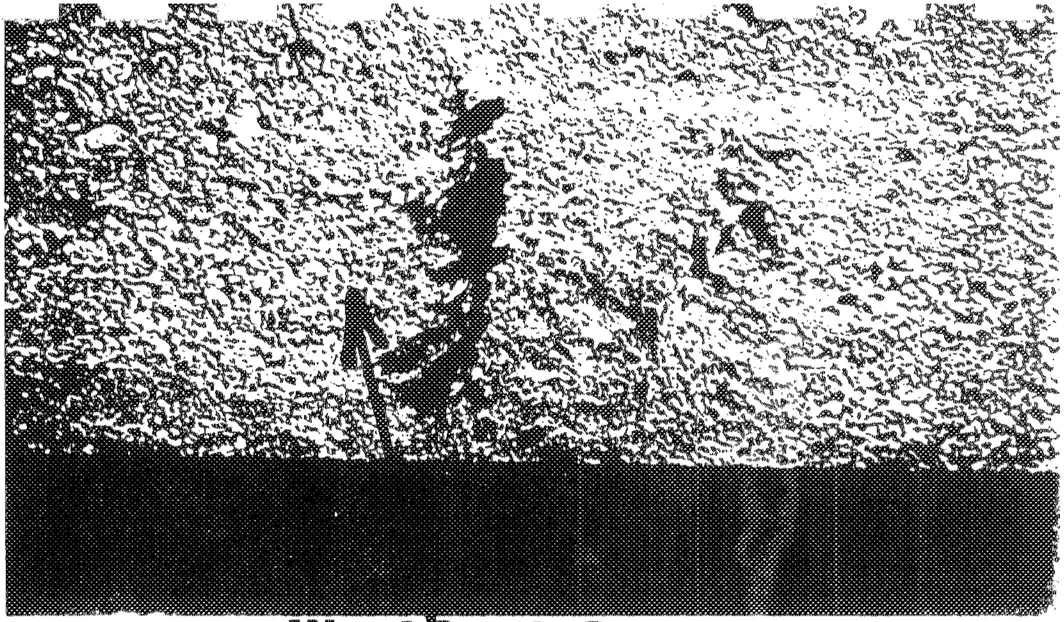
East Levee@124+50
Open outboard Slope



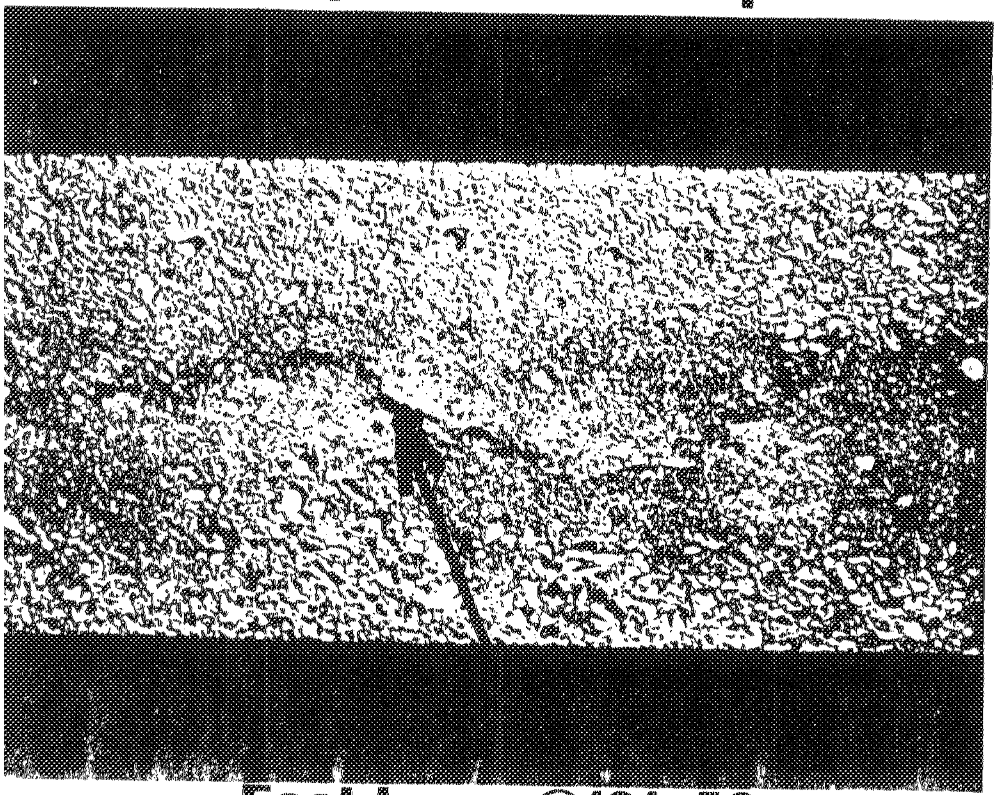
West Levee@132+00
Animal hole



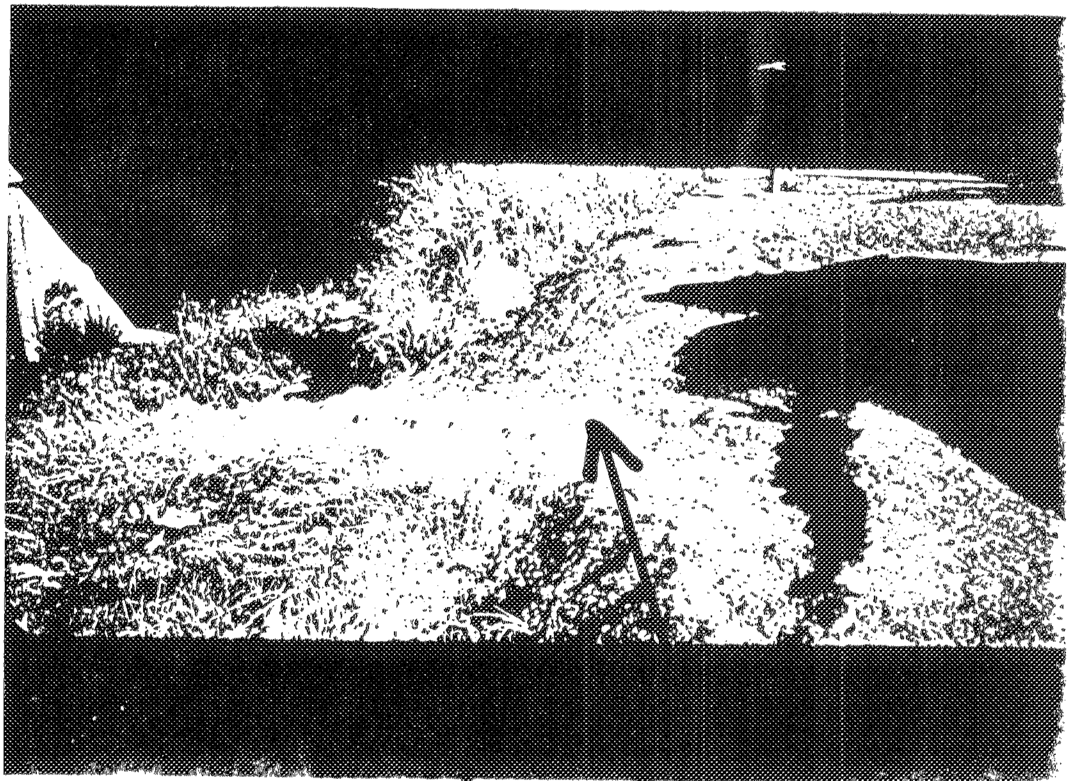
West Levee@134+00
Open inboard slope



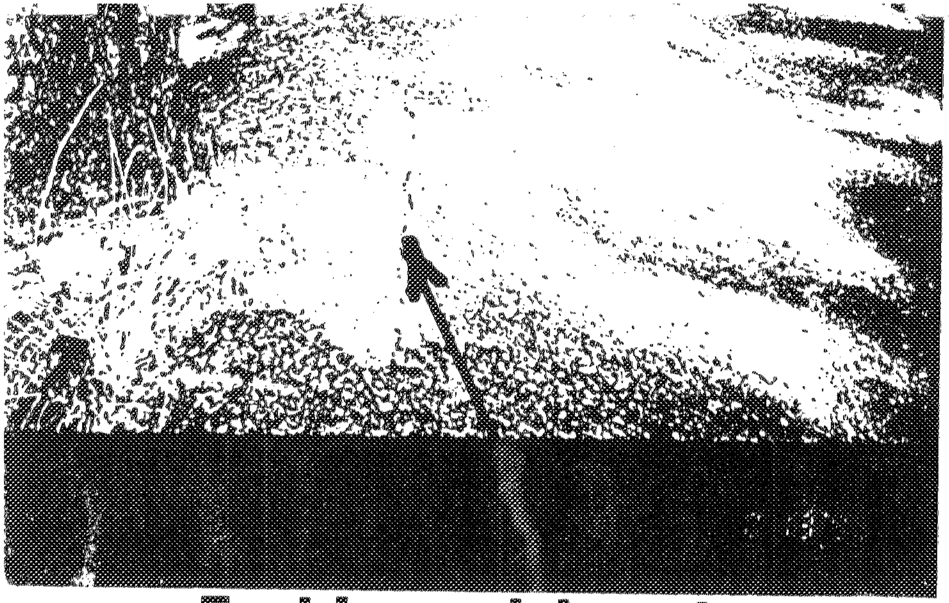
West Lest @ 131+00
Open Test Pit?



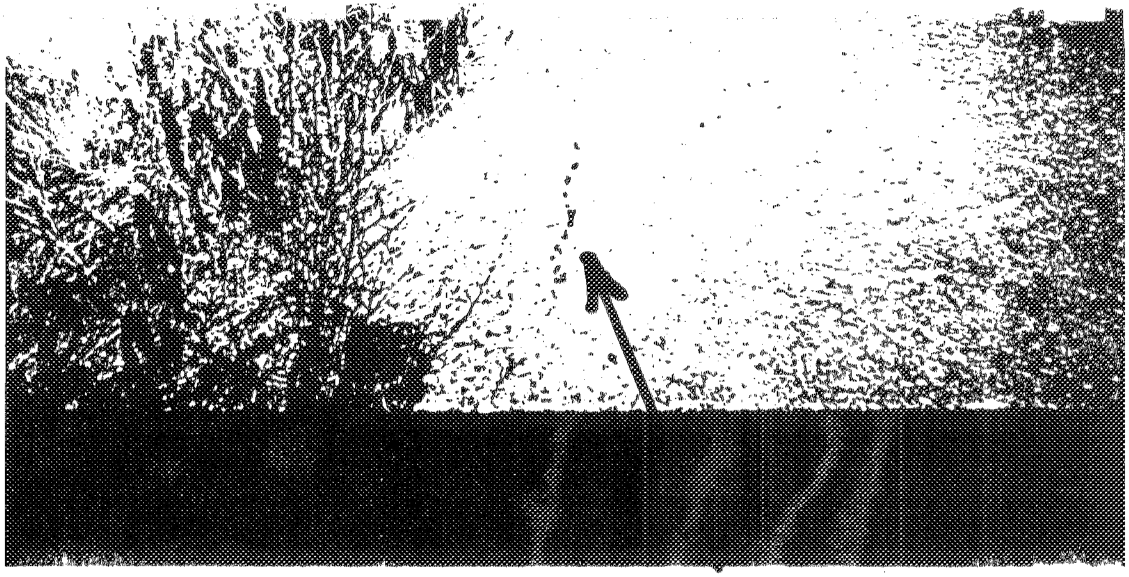
East Levee@121+50
Tension/Dessication Crack



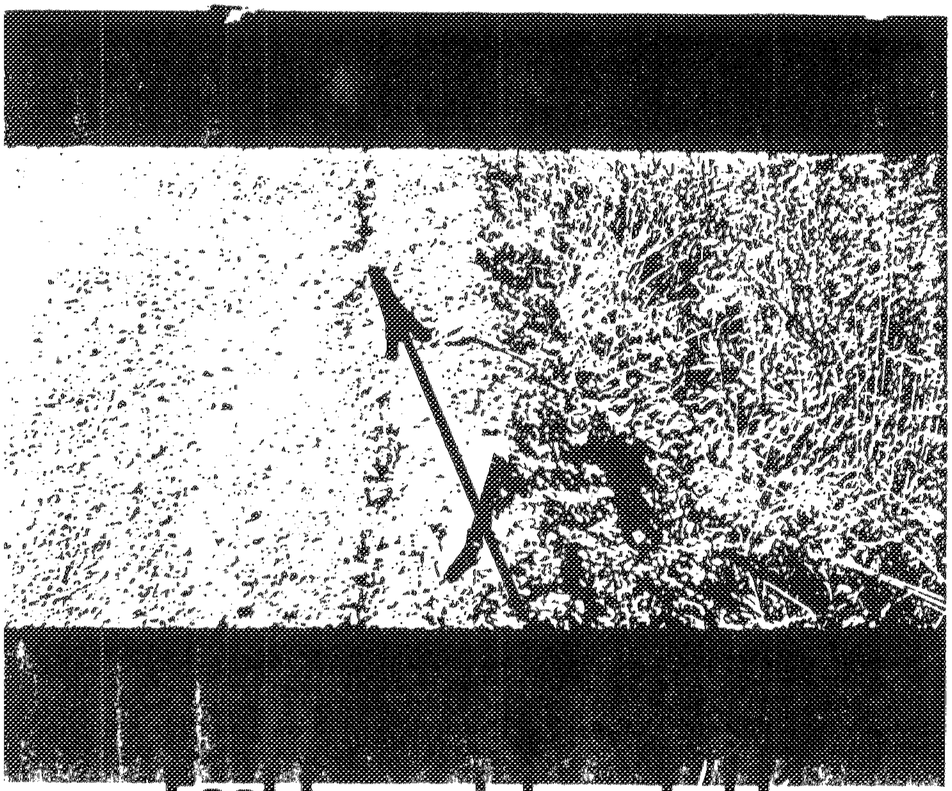
West Levee@136+80
Slope erosion near Trimble Rd.Bridge



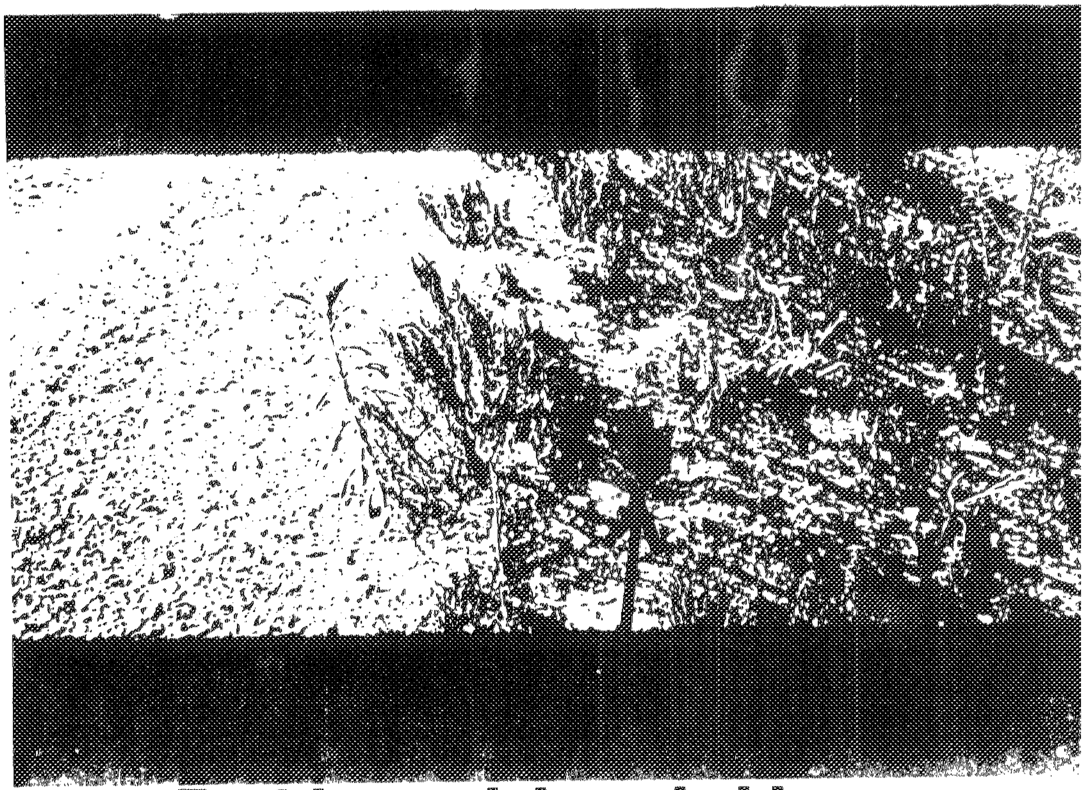
East Levee-inboard side
Crack @ 133+00



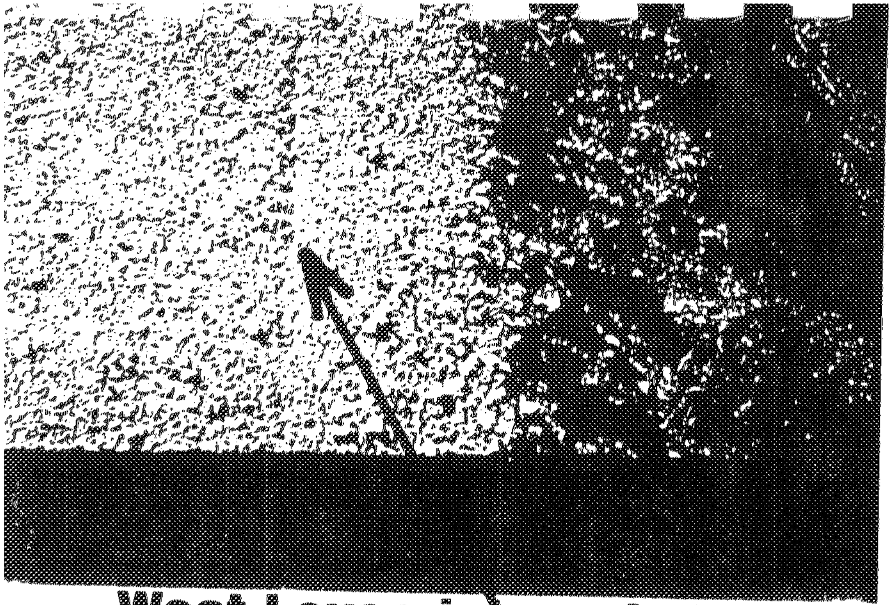
East Levee-inboard side
Crack @ 126+20



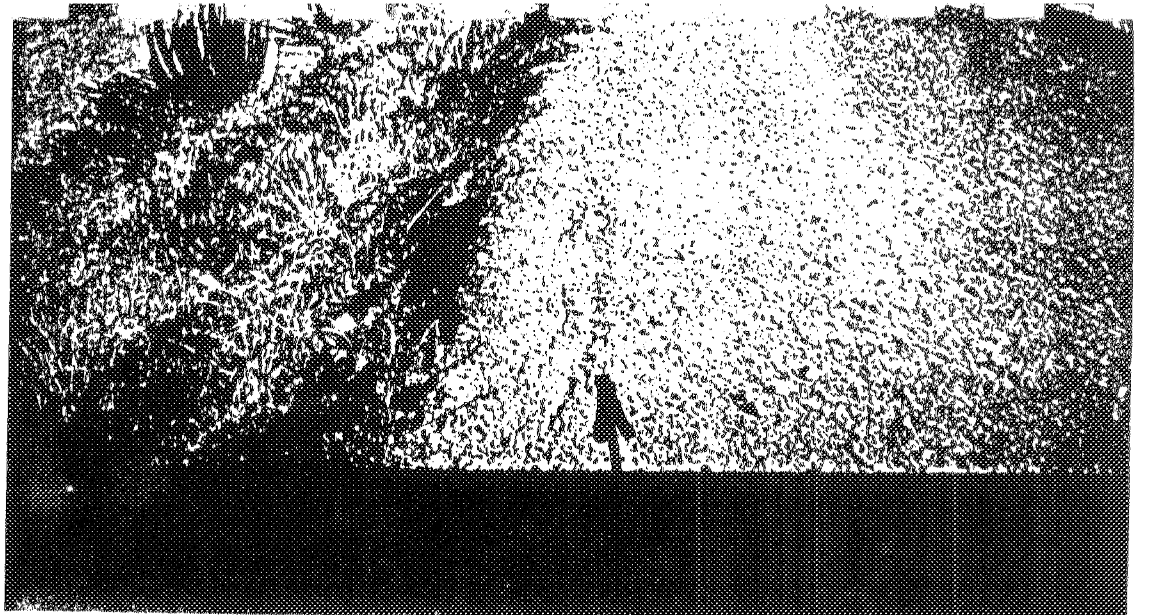
East Levee-inboard side
Crack @ 126+00



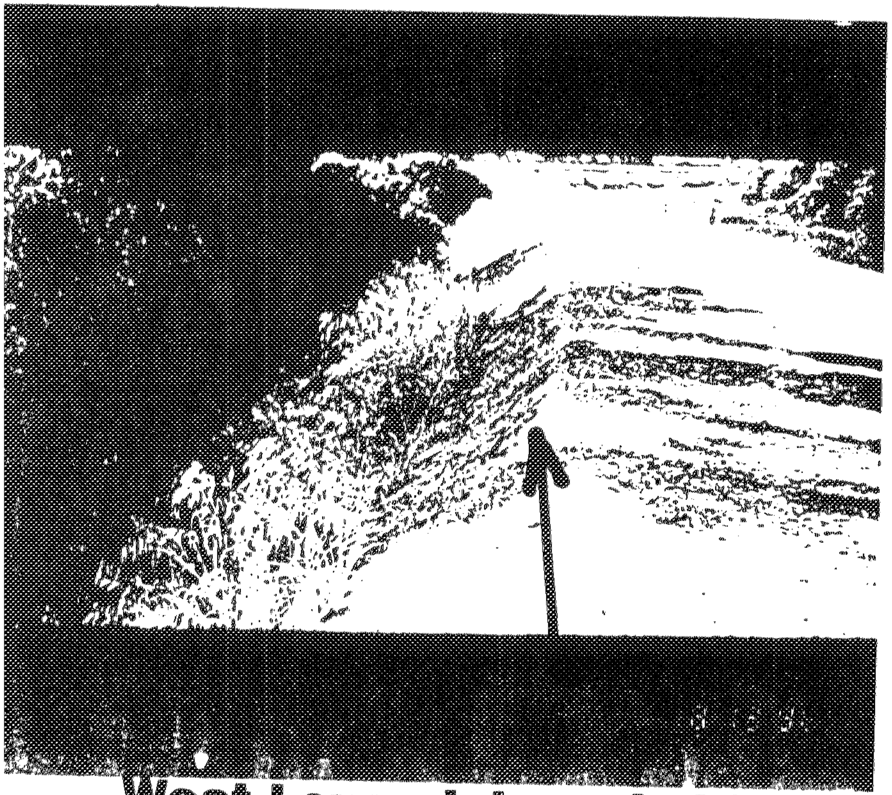
East Levee-inboard side
Animal burrow @ 124+00



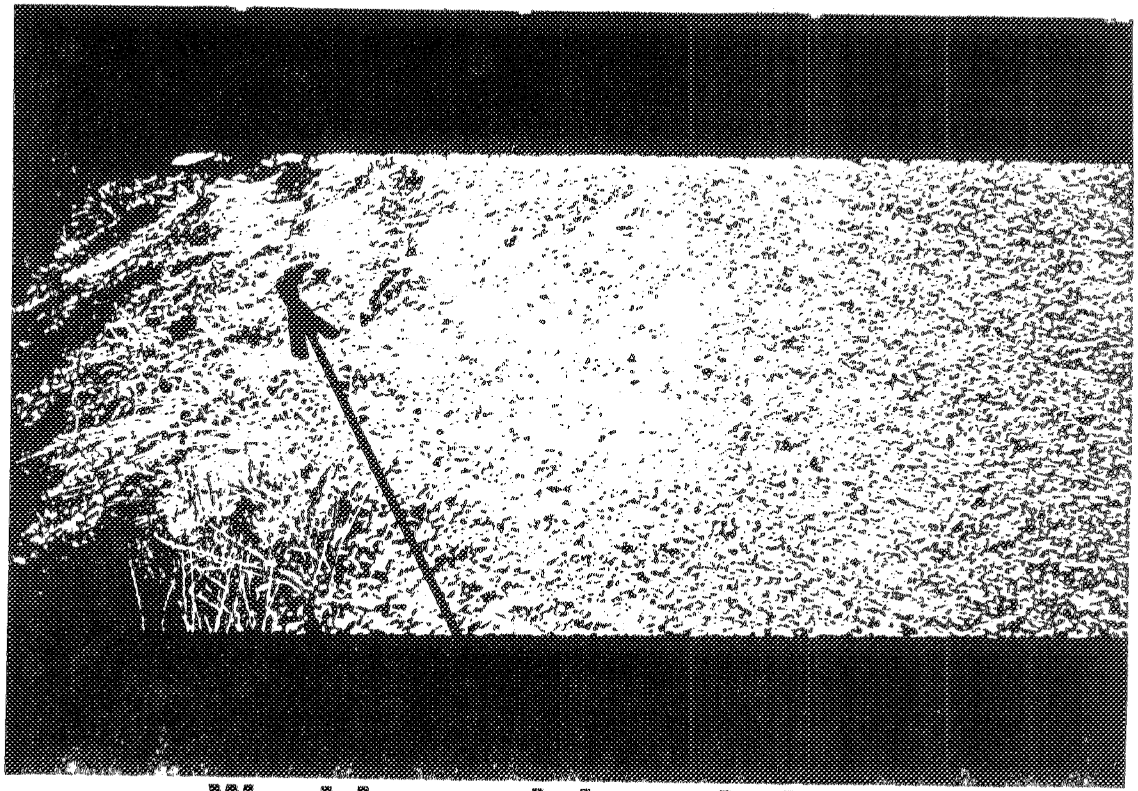
West Levee-inboard side
Crack @ 128+00



West Levee-inboard side
Crack @ 127+50

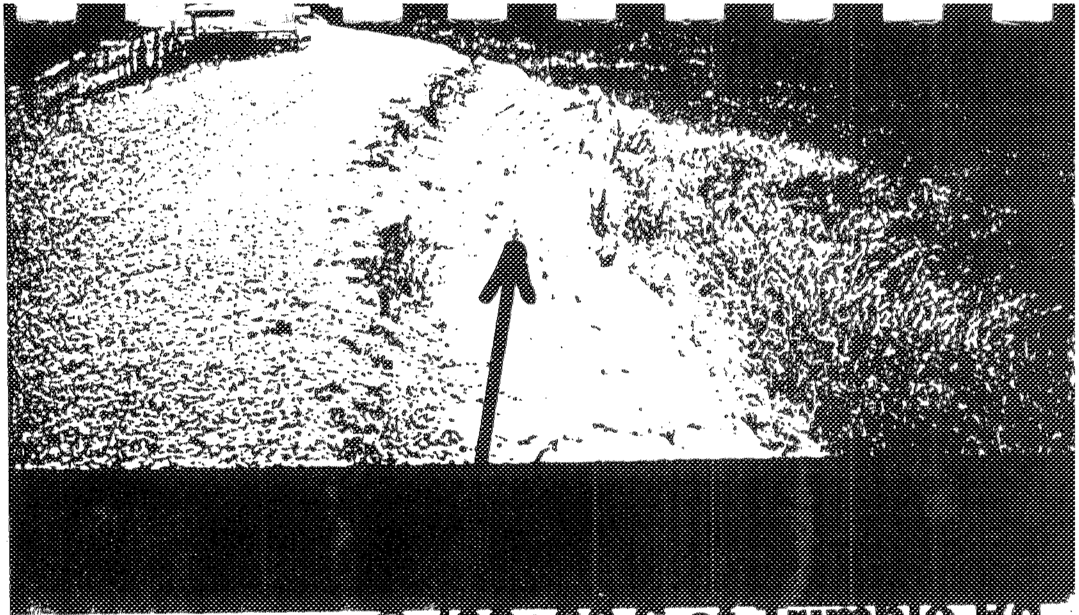


West Levee-inboard side
Steep slope @ 129+00



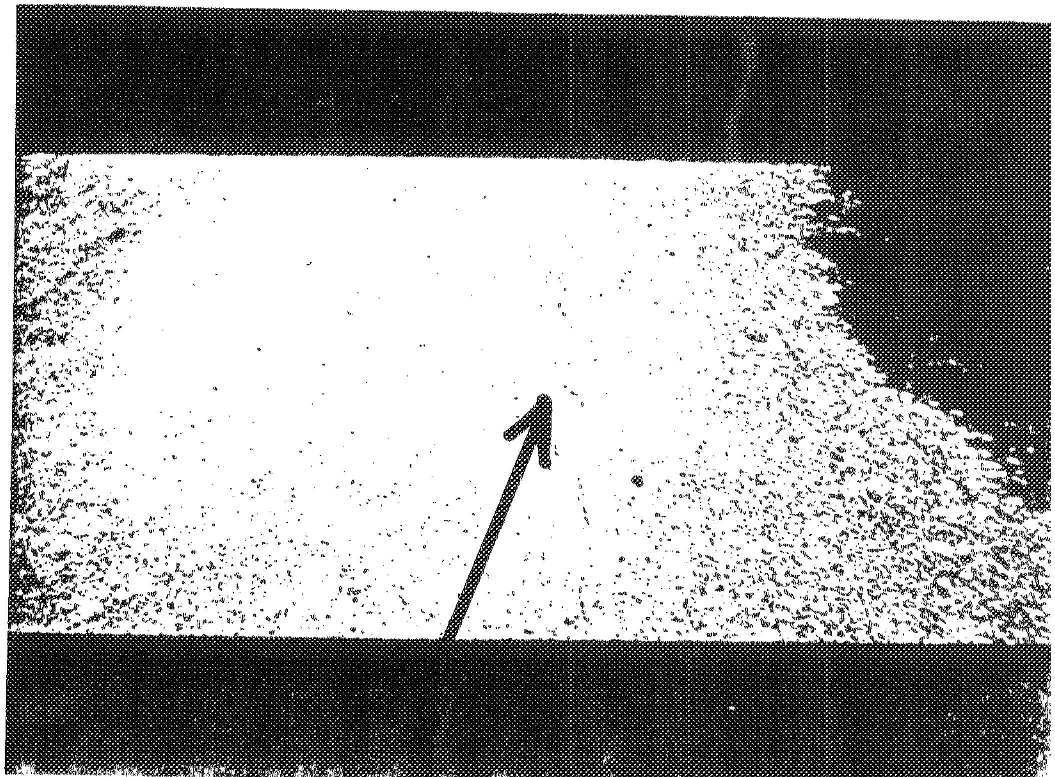
West Levee-inboard side
Animal burrow/slope erosion @ 130+00

East Levee @ 137+00, S.of Trimble Rd.
Open Steep Slope

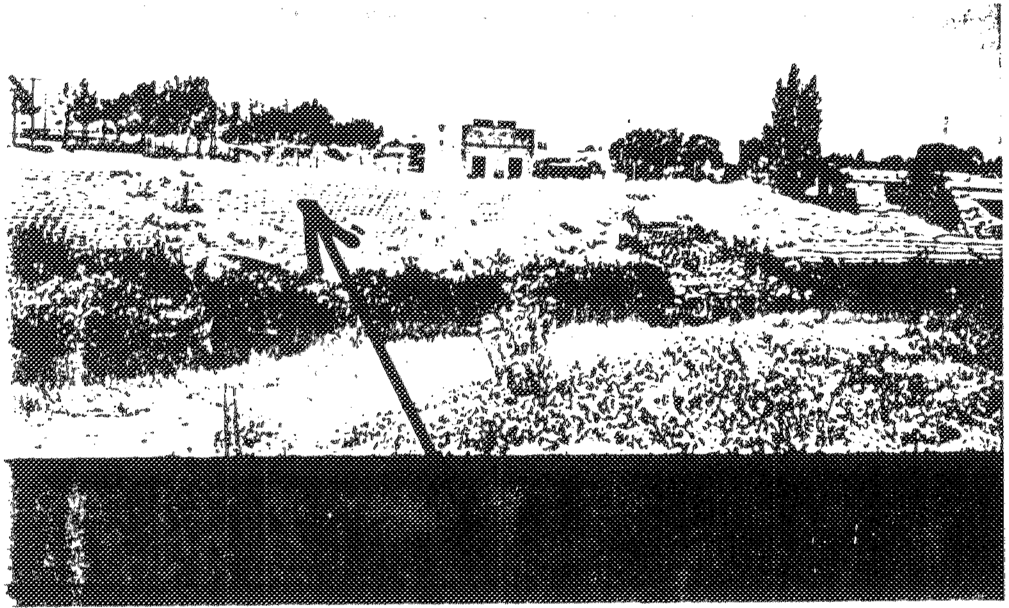


East Levee @ 138+00,S.of Trimble Rd.
Sand bag slope protechtion

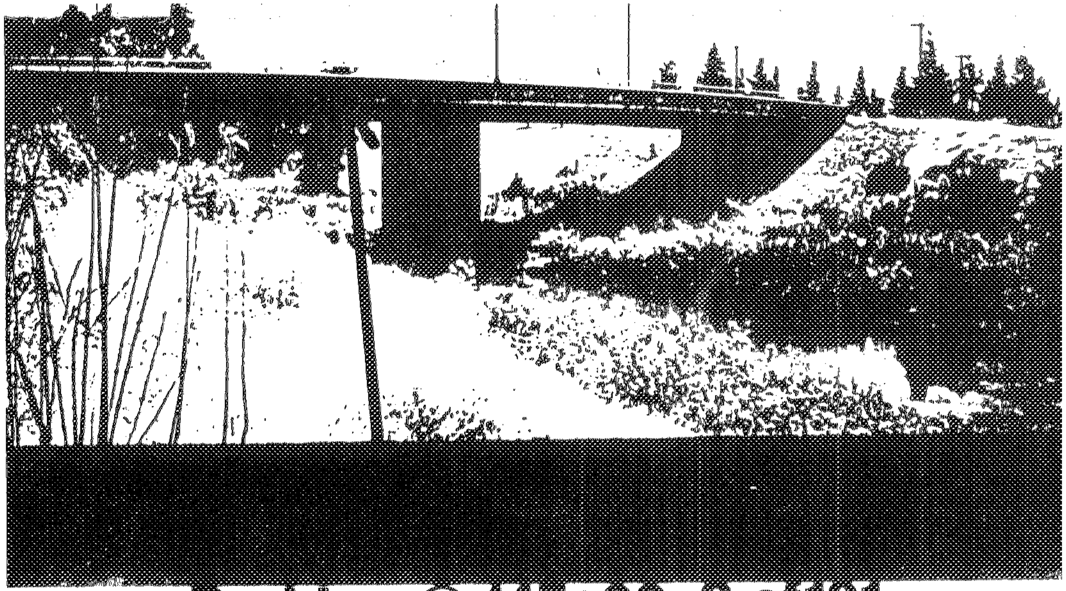
East Levee @ 144+50,N.of101
Concrete lining on inboard slope



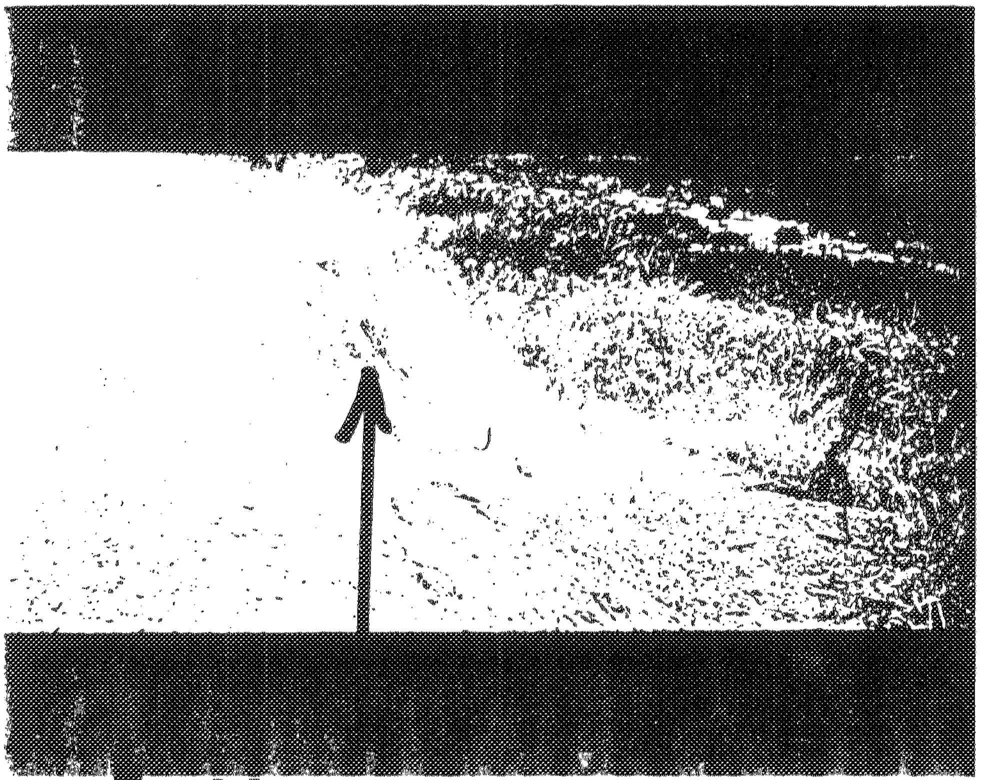
East Levee @ 148+00,S.of101
Tension crack



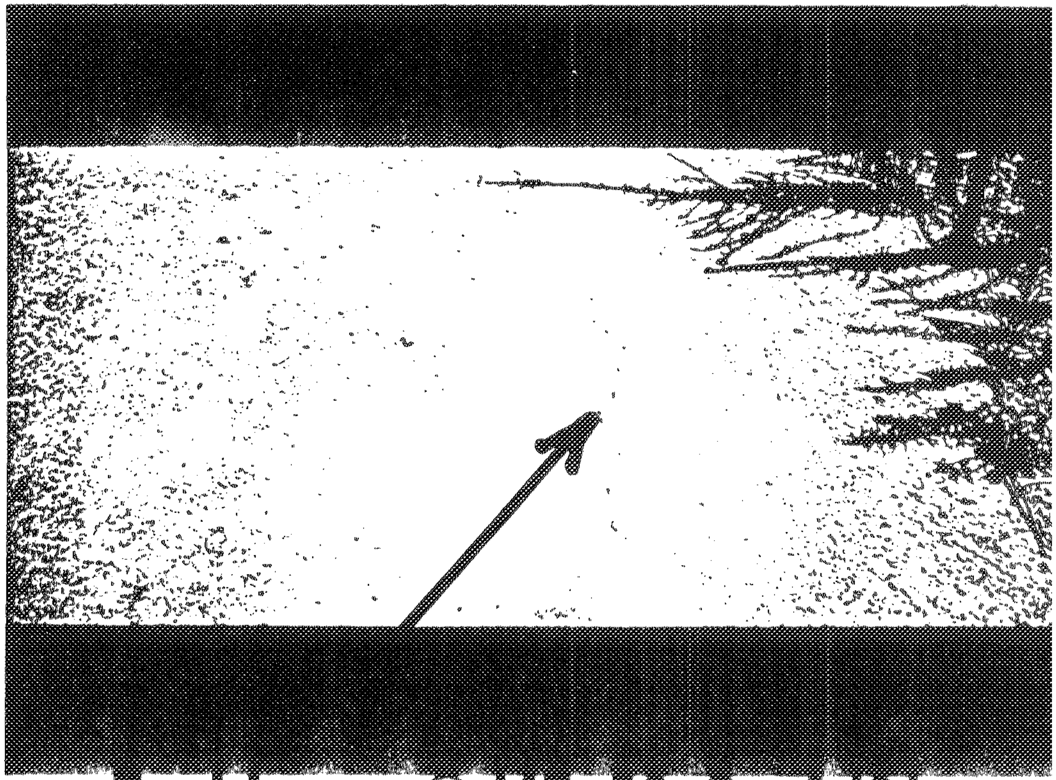
West Levee,S of 101 @ 145+50
Sand bag slope protection



Breidge @ 147+00, S.of101
Connecting Long-Term Parking



East Levee @ 144+50, N. of 101
Crack on open inboard slope



East Levee @ 147+50, S. of 101
Tension crack

APPENDIX-C

District's Specifications, Field Density Test Results, and Atterberg's Limit Test Results

Corporate Offices
Materials/Chemistry Laboratories
415 Fairchild Drive
Mountain View, California 94043
Telephone: (415) 967-6982
Facsimile: (415) 967-6955

DCI
DYNAMIC CONSULTANTS, INC.
Testing & Inspection Services

Branch Office
26 Hangar Way, Suite F
Watsonville, California 95076
Telephone: (408) 724-2234
Facsimile: (408) 724-9166

August 16, 1995/dmj

Santa Clara Valley Water District
5750 Almaden Expressway
San Jose, California 95118

DCI No.: 5531-M01
P.O. No.: 960030548
Lab No.: 5-524358

Attention: Ann Cooper

PROJECT: *Guadalupe River Levee Raising*

SUBJECT: Laboratory Test Results

SAMPLE: Five (5) samples of import fill were submitted to our laboratory by your representative on August 3, 1995 and were tested in accordance with ASTM D4318.

SUMMARY OF TEST RESULTS

Plasticity Index - ASTM D4318

Sample Number	*Liquid Limit*	*Plastic Limit*	*Plasticity Index*	*Group Symbol*
1	58.0	25.8	32.2 or 32	CH
2	50.8	22.4	28.4 or 28	CH / CL
3	58.7	27.9	30.8 or 31	CH
4	73.7	34.4	39.3 or 39	CH / MH
5	25.5	15.3	10.2. or 10	CL

Sample Location:

No. 1 - 200' S. Doyle - PGE Tower
No. 2 - Stockpile - east of line C - Sta. 35+00
No. 3 - 200' east, 150' North of Asbury
No. 4 - 150' S. Hedding, 150' E. Irene
No. 5 - 150' S. Hobson, W. Bank

Respectfully submitted,
DYNAMIC CONSULTANTS, INC.

Denny Zucchi

Denny Zucchi
Laboratory Supervisor
2c:

Corporate Offices
Materials/Chemistry Laboratories
415 Fairchild Drive
Mountain View, California 94043
Telephone: (415) 967-6982
Facsimile: (415) 967-6955

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Testing & Inspection Services

Branch Office
26 Hangar Way, Suite F
Watsonville, California 95076
Telephone: (408) 724-2234
Facsimile: (408) 724-9166

August 18, 1995/dmj

Santa Clara Valley Water District
5750 Almaden Expressway
San Jose, California 95118

DCI No.: 5531-M01
P.O. No.: 960030548
Lab No.: 5-524379

Attention: Ann Cooper

PROJECT: ***Guadalupe River Levee Raising***

SUBJECT: Laboratory Test Results

SAMPLE: Three (3) samples of import fill were sampled by our representative and were tested in accordance with ASTM Designations D1557 and D4318.

SUMMARY OF TEST RESULTS

A. *Maximum Density Curve* - ASTM D1557

Sample Number	*Maximum Dry Density, pcf*	*Optimum Moisture, %*
3	111.2	16.2
4	108.2	18.2
5	101.5	19.8

B. *Plasticity Index* - ASTM D4318

Sample Number	*Liquid Limit*	*Plastic Limit*	*Plasticity Index*	*Group Symbol*
4	64.6	27.0	37 or 38	CH

Respectfully submitted,
DYNAMIC CONSULTANTS, INC.

Denny Zucchi

Denny Zucchi
Laboratory Supervisor

8/28/95

2c:

UNITED SOIL ENGINEERING. INC.
Soil. Foundation and Geological Engineers
3476 EDWARD AVENUE. SANTA CLARA. CALIFORNIA 95054 (408) 988-2990

SANTA CLARA
VALLEY WATER DISTRICT
95 JUL 17 AM 11:04

File No. 95-4157-S1
July 6, 1995

Ferma Corporation
6655 Smith Avenue
Newark, CA 94560

Attention: Mr. Larry Mosbey

Subject: Guadalupe River Levee Improvement
From Trimble Road to Highway 101
San Jose, California
TESTING AND INSPECTION SERVICES

Dear Mr. Mosbey:

Pursuant to your request, we are transmitting herewith the results of our testing and inspection services for the Guadalupe River Levee Improvement section located between Trimble Road and Highway 101 in San Jose, California.

The improvement of the existing levees section from Trimble Road to Highway 101 consisted of raising the existing levee to approximately 8 inches of import material and 3 inches of baserock. The import material and baserock were compacted to a minimum of 90% and 95% maximum relative compaction according to test procedure ASTM D1557-85. The grading operation started on June 22 and ended on June 28, 1995. Based on the result of field moisture and density tests, we have certified that the grading contractor (Ferma Corporation) has performed the grading operation to our satisfaction.

The result of field moisture and density tests are summarized in Table I. The compaction curves of the import material and the baserock are shown in Figures 1 and 2.

If you have any questions or require additional information, please feel free to contact our office at your convenience.

Very truly yours,

UNITED SOIL ENGINEERING, INC.

Vien Vo, P.E.

use-4425/Copies: 2 to Ferma Corporation

TABLE I

SUMMARY OF FIELD DENSITY TEST RESULTS

Test No.	Date 1995	Station No.	Moisture % Dry Wt.	Dry Density p.c.f.	Rel. Comp % of Max.	Soil Type & Remark
		WEST LEVEE - IMPORT MATERIAL				
1	6/22	270 + 00	18.7	105.0	92.1	A
2	"	269 + 00	18.3	104.5	91.7	A
3	"	268 + 00	19.1	104.3	91.5	A
4	"	267 + 00	17.9	106.5	93.4	A
5	"	266 + 00	18.8	107.1	93.9	A
6	"	265 + 00	19.5	105.7	92.7	A
7	6/23	264 + 00	19.4	106.0	93.0	A
8	"	263 + 00	18.5	105.5	92.5	A
9	"	262 + 00	19.7	106.7	93.6	A
10	"	261 + 00	20.0	107.3	94.1	A
11	"	260 + 00	19.3	106.6	93.5	A
12	"	259 + 00	18.9	105.8	92.8	A
13	"	258 + 00	17.5	107.5	94.3	A

File No. 95-4157-S2

TABLE I

SUMMARY OF FIELD DENSITY TEST RESULTS

Test No.	Date 1995	Station No.	Moisture % Dry Wt.	Dry Density p.c.f.	Rel. Comp % of Max.	Soil Type & Remark
EAST LEVEE - IMPORT MATERIAL						
14	6/26	270 + 00	18.3	106.0	93.0	A
15	"	269 + 00	17.4	107.2	94.0	A
16	"	268 + 00	17.9	106.8	93.7	A
17	"	267 + 00	18.7	107.7	94.5	A
18	"	266 + 00	18.3	105.8	92.8	A
19	"	265 + 00	17.3	106.4	93.3	A
20	"	264 + 00	18.0	105.5	92.5	A
21	"	263 + 00	17.7	105.0	92.1	A
22	"	262 + 00	18.5	106.3	93.2	A
23	"	261 + 00	18.6	107.1	93.9	A
24	"	260 + 00	18.3	106.8	93.7	A
25	"	259 + 00	17.9	106.5	93.4	A
26	"	258 + 00	17.6	107.2	94.0	A

e No. 95-4157-S2

TABLE I

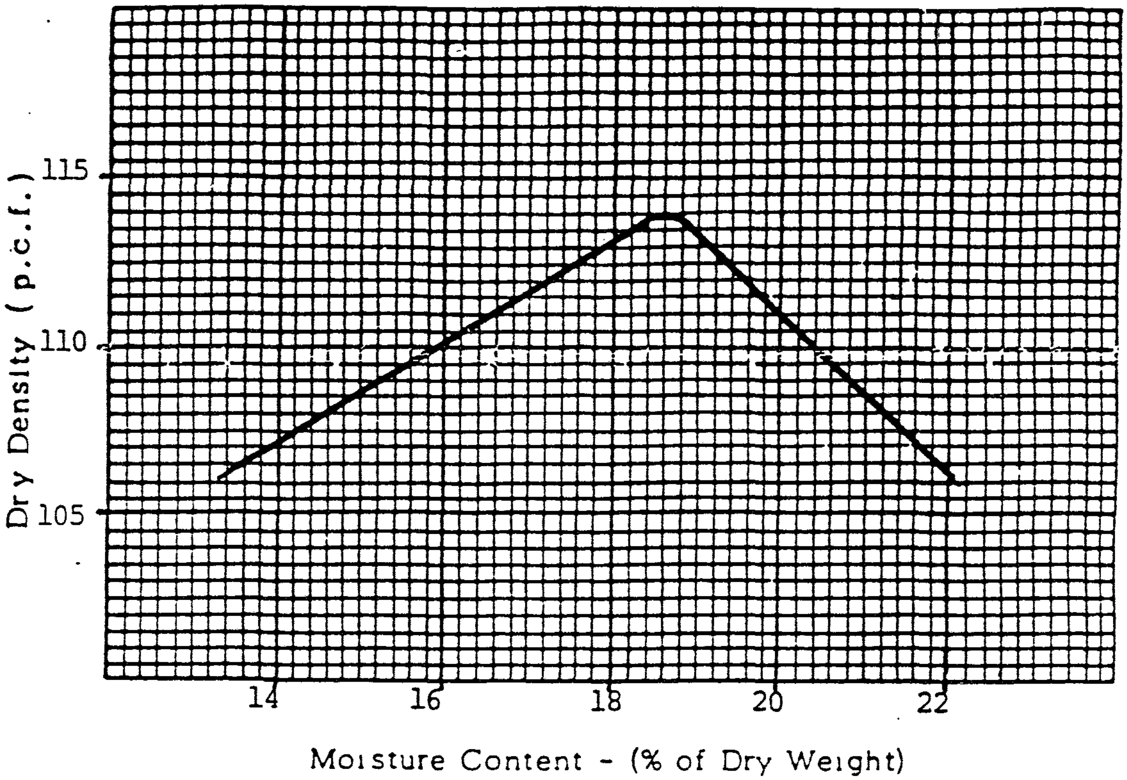
SUMMARY OF FIELD DENSITY TEST RESULTS

Test No.	Date 1995	Station No.	Moisture % Dry Wt.	Dry Density p.c.f.	Rel. Comp % of Max.	Soil Type & Remark
			WEST LEVEE - BASEROCK			
27	6/28	270 + 00	6.5	130.0	96.3	B
28	"	269 + 00	6.3	128.4	95.1	B
29	"	268 + 00	6.7	129.0	95.5	B
30	"	267 + 00	5.8	129.2	95.7	B
31	"	266 + 00	6.1	129.5	95.9	B
32	"	265 + 00	5.9	129.1	95.6	B
33	"	264 + 00	6.0	128.5	95.2	B
34	"	263 + 00	6.3	128.8	95.4	B
35	"	262 + 00	5.5	130.7	96.8	B
36	"	261 + 00	5.7	129.3	95.8	B
37	"	260 + 00	6.4	129.5	95.9	B
38	"	259 + 00	6.5	130.1	96.4	B

TABLE I

SUMMARY OF FIELD DENSITY TEST RESULTS

Test No.	Date 1995	Station No.	Moisture % Dry Wt.	Dry Density p.c.f.	Rel. Comp % of Max.	Soil Type & Remark
			EAST LEVEE - BASEROCK			
39	6/28	270 + 00	6.0	128.8	95.4	B
40	"	269 + 00	5.8	128.7	95.3	B
41	"	268 + 00	6.1	129.5	95.9	B
42	"	267 + 00	5.5	129.7	96.1	B
43	"	266 + 00	5.7	130.1	96.4	B
44	"	265 + 00	5.5	128.9	95.5	B
45	"	264 + 00	6.2	129.3	95.8	B
46	"	263 + 00	6.0	129.5	95.9	B
47	"	262 + 00	5.7	128.8	95.4	B
48	"	261 + 00	6.1	130.3	96.5	B
49	"	260 + 00	5.8	129.6	96.0	B
50	"	259 + 00	5.6	128.5	95.2	B



SAMPLE: A

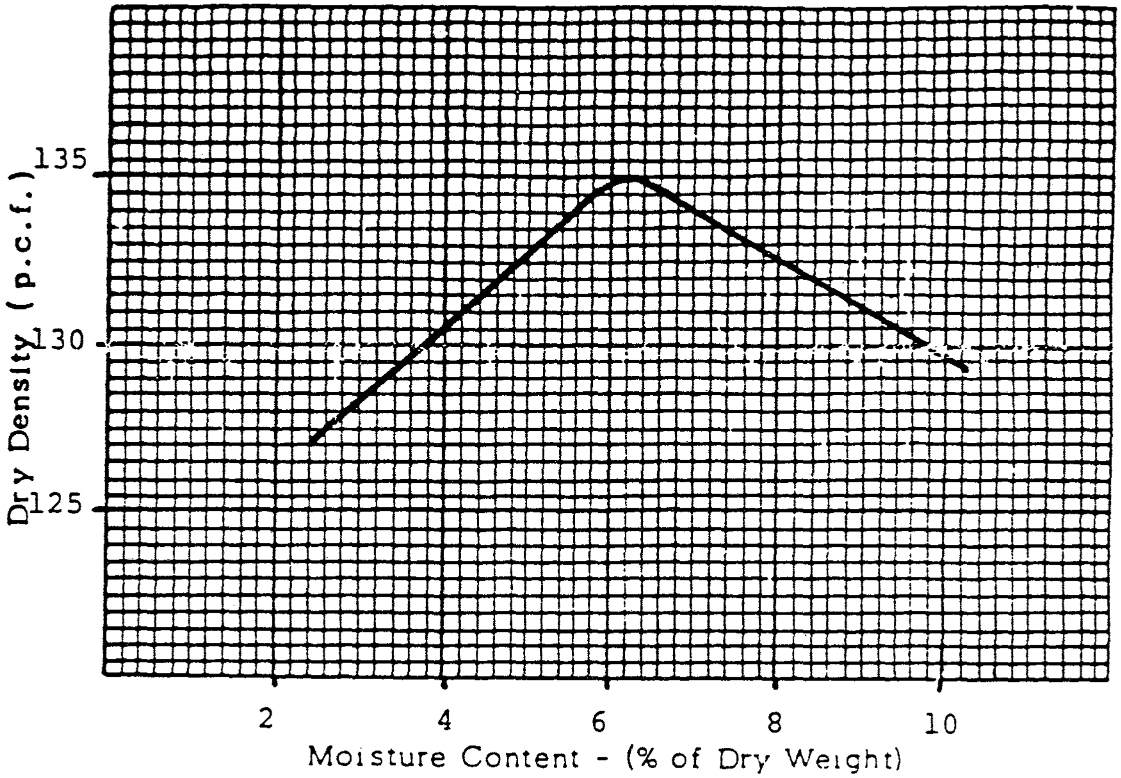
DESCRIPTION: Brown to reddish brown sandy CLAY (import)

LABORATORY TEST PROCEDURE: ASTM D1557-85

MAXIMUM DRY DENSITY: 114.0 p.c.f.

OPTIMUM MOISTURE CONTENT: 18.6%

igure 1 - Compaction Curve A



SAMPLE: B

DESCRIPTION: Gray Baserock

LABORATORY TEST PROCEDURE: ASTM D1557-85

MAXIMUM DRY DENSITY: 135.0 p.c.f.

OPTIMUM MOISTURE CONTENT: 6.2%

Figure 2 - Compaction Curve B

Corporate Offices
Materials/Chemistry Laboratories
415 Fairchild Drive
Mountain View, California 94043
Telephone: (415) 967-6982
Facsimile: (415) 967-6955

DCI

DYNAMIC CONSULTANTS, INC.

Testing & Inspection Services

Branch Office
26 Hangar Way, Suite F
Watsonville, California 95076
Telephone: (408) 724-2234
Facsimile: (408) 724-9166

October 27, 1995/al

LAB NO.: 36273

DCI No.: 5531-M01
P.O. No.: 960030548

REPORT TO: Santa Clara Valley Water District
5750 Almaden Expressway
San Jose, California 95118

Attention: Ann Cooper

PROJECT: GUADALUPE RIVER LEVEE RAISING

SERVICE: SOIL COMPACTION TESTS

10/23/95 -- 4.00 Hrs. S.T. -- Insp.: Joe Matteucci

Performed fourteen (14) field density tests.

TEST NO.	DESCRIPTION	ELEVATION	MAX. DENS. LBS./CU. FT.	MOIST. %	DRY DENS. LBS./CU. FT.	REL. COMP. %	REQD. REL. COMP. %
1 - 10/23	East Levee, Montague Expressway end to Hetch Hetchy	Finish Subgrade	134.5	13.0	123.5	92	90
2 - 10/23	East Levee, Montague Expressway end to Hetch Hetchy	Finish Subgrade	134.5	11.3	120.8	90	90
3 - 10/23	East Levee, Montague Expressway end to Hetch Hetchy	Finish Subgrade	134.5	11.0	124.0	92	90
4 - 10/23	East Levee, Montague Expressway end to Hetch Hetchy	Finish Subgrade	134.5	12.4	120.5	90	90
5 - 10/23	East Levee, Montague Expressway end to Hetch Hetchy	Finish Subgrade	134.5	13.8	121.8	91	90
6 - 10/23	East Levee, Tasman Drive to Hetch Hetchy	Finish Subgrade	125.5	13.5	112.3	90	90
7 - 10/23	East Levee, Tasman Drive to Hetch Hetchy	Finish Subgrade	125.5	13.4	113.8	91	90
8 - 10/23	East Levee, Tasman Drive to Hetch Hetchy	Finish Subgrade	125.5	12.6	118.3	94	90
9 -10/23	East levee, between Hetch Hetchy to end of concrete retaining wall to Tasman Drive*	Finish Subgrade	142.3	9.9	133.5	94	90
10 - 10/23	East levee, between Hetch Hetchy to end of concrete retaining wall to Tasman Drive	Finish Subgrade	142.3	9.3	138.1	97	90

10:[illegible]

Page Two (2) of Two (2)
DCI No.: 5531-M01
Lab No.: 36273
October 27, 1995/al

Santa Clara Valley Water District

10/23/95 - Continued

TEST NO.	DESCRIPTION	ELEVATION	MAX. DENS. LBS./CU. FT.	MOIST. %	DRY DENS. LBS./CU. FT.	REL. COMP. %	REQD. REL. COMP. %
11 - 10/23	East levee, between Hetch Hetchy to end of concrete retaining wall to Tasman Drive*	Finish Subgrade	142.3	8.0	133.2	94	90
12 - 10/23	East levee, between Hetch Hetchy to end of concrete retaining wall to Tasman Drive*	Finish Subgrade	142.3	8.4	136.3	96	90
13 - 10/23	East levee, between Hetch Hetchy to end of concrete retaining wall to Tasman Drive*	Finish Subgrade	141.5	11.7	127.2	90	90
14 - 10/23	East levee, between Hetch Hetchy to end of concrete retaining wall to Tasman Drive*	Finish Subgrade	141.5	11.7	133.5	94	90

***NOTE:** *Additional tests taken in same areas tested on October 19, 1995 and October 23, 1995*

Respectfully submitted,
DYNAMIC CONSULTANTS, INC.
Clifford N. Craig

Clifford N. Craig, P.E.
Senior Engineer

15.14. Earthfill Material

Earthfill material shall consist of furnishing, transporting, and placing fill material at levees within Santa Clara County. These sites may include but are not limited to:

- Guadalupe River between Highway 101 and Gold Street
- San Tomas Creek between Highways 237 and 101
- Canoas Creek at various sites between Nightingale Drive and Dow Drive
- Coyote Creek from Montague Expressway to Charcot Avenue

Contractor shall be capable of furnishing, transporting, and placing and compacting 2,000 tons of earthfill per day.

Imported borrow material shall conform to Section 19-7.01 of the State Specifications.

Material shall be free of deleterious or organic material, pervious layers, and of debris.

The earthfill material shall be of the type and quality suitable for embankment construction. The Contractor shall provide test results for the Plasticity Index and gradation of the earthfill material for approval by the Engineer prior to importing and placing the material on the project site. The imported borrow material shall be free of organic contents and conform to the following requirements:

Plasticity Index	25 Maximum 10 Minimum

The Contractor shall not import earthfill material that is contaminated with regulated materials. The Contractor shall submit to the Engineer for review and acceptance an Imported Materials Certification Form for each source of imported earthfill material.

If the Contractor uses earthfill materials that are, or are found to be, contaminated by regulated materials, the Contractor shall immediately remove the contaminated material and dispose of it in accordance with all applicable laws, ordinances, and regulations; conduct necessary sampling and monitoring to verify that all contaminated material has been removed, and verify to the satisfaction of the District and/or appropriate regulatory agencies that any surrounding areas, materials, soils or waters have not been impacted by the contaminated materials. The subsequent disposal of the contaminated material shall be the sole responsibility of the Contractor. No additional compensation will be made to the Contractor by the District for removal, disposal, replacement, chemical analysis, or any other costs associated with the removal, disposal, and replacement of the contaminated material.

Weight of material shall be determined by scales to be furnished by the Contractor.

Full compensation for furnishing, delivery and placement of fill material at designated work sites shall be included in the unit price bid per ton for EARTHFILL MATERIAL, Bid item No. 41.

APPENDIX-D

Site Plan Showing Previous Boring Locations, Idealized Soil Profile,
and Land Subsidence Records

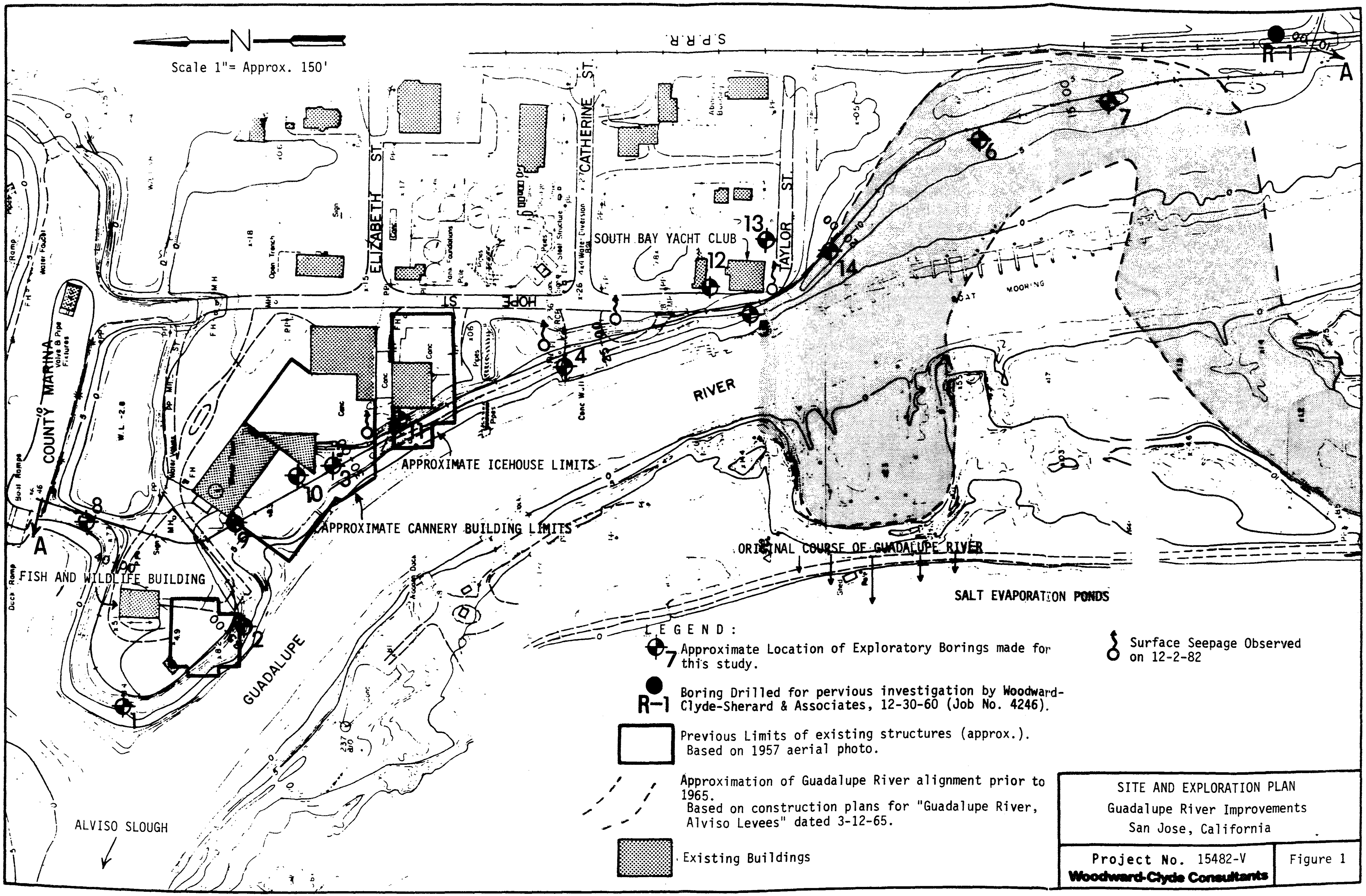
Guadalupe River

Gold St

Lafayet St

	1960	1961	1963	1964	1965	1966	1967	1968	1969	1970	1971	1972	1973	1974	1975	1976	1979	1983	1988	1989
X555 (ft)	1.09	0.83	0.08	-0.17	-0.53	-0.88	-1.29	-1.54		-1.76	-1.69	-1.70	-1.70	-1.68	-1.66	-1.65				
NGVD29 m	0.332	0.253	0.024	-0.052	-0.162	-0.268	-0.393	-0.469		-0.536	-0.515	-0.518	-0.518	-0.512	-0.506	-0.503				
88 m (+0.79)	1.122	1.043	0.814	0.738	0.628	0.522	0.397	0.321		0.254	0.275	0.272	0.272	0.278	0.284	0.287				
Y555 (ft)	3.24	2.99	2.25	1.99	1.65	1.29	0.90	0.64	0.45	0.41	0.48	0.47								
NGVD29 m	0.988	0.911	0.686	0.607	0.503	0.393	0.274	0.195	0.137	0.125	0.146	0.143								
88 m (+0.79)	1.778	1.701	1.476	1.397	1.293	1.183	1.064	0.985	0.927	0.915	0.936	0.933								
F555 (ft)	7.88						7.11													
NGVD29 m	2.402						2.167													
88 m (+0.81)	3.212						2.977													
J555 (ft)	5.93	5.92		5.28	5.17	4.96	4.57	4.38	4.25	4.19	4.20	4.33		4.55	4.60					
NGVD29 m	1.807	1.804		1.609	1.576	1.512	1.393	1.335	1.295	1.277	1.280	1.320		1.387	1.402					
88 m (+0.80)	2.607	2.604		2.409	2.376	2.312	2.193	2.135	2.095	2.077	2.080	2.120		2.187	2.202					
K555 (ft)		3.39		2.51	2.28	1.97	1.56	1.27	1.13	1.06	1.11	1.17		1.58	1.62					
NGVD29 m		1.033		0.765	0.695	0.600	0.475	0.387	0.344	0.323	0.338	0.357		0.482	0.494					
88 m (+0.79)		1.823		1.555	1.485	1.390	1.265	1.177	1.134	1.113	1.128	1.147		1.272	1.284					
TBM31 (ft)	12.87		11.79	11.54	11.19	10.81	10.51		10.00	9.98	10.05	9.98								
NGVD29 m	3.923		3.594	3.517	3.411	3.295	3.203		3.048	3.042	3.063	3.042								
88 m (+0.81)	4.733		4.404	4.327	4.221	4.105	4.013		3.858	3.852	3.873	3.852								
Tidal 2 (ft)					13.76	13.33	12.95	12.66	12.46	12.42	12.47	12.42	12.46	12.42	12.43	12.44		12.24		
NGVD29 m					4.194	4.063	3.947	3.859	3.798	3.786	3.801	3.786	3.798	3.786	3.789	3.792		3.731		
88 m (+0.81)					5.004	4.873	4.757	4.669	4.608	4.596	4.611	4.596	4.608	4.596	4.599	4.602		4.541		
Q876 (ft)	22.99						19.63													
NGVD29 m	7.007						5.983													
88 m (+0.81)	7.817						6.793													
B112 (ft)	75.77						72.67												71.92	
NGVD29 m	23.095						22.150												21.921	
88 m (+0.84)	23.935						22.990												22.761	
F175 (ft)	82.91						79.55										79.09		78.85	78.81
NGVD29 m	25.271						24.247										24.107		24.034	24.021
88 m (+0.84)	26.111						25.087										24.947		24.874	24.861

LAND SUBSIDENCE RECORD



LEGEND:

- Approximate Location of Exploratory Borings made for this study.
- R-1 Boring Drilled for pervious investigation by Woodward-Clyde-Sherard & Associates, 12-30-60 (Job No. 4246).
- Previous Limits of existing structures (approx.). Based on 1957 aerial photo.
- Approximation of Guadalupe River alignment prior to 1965. Based on construction plans for "Guadalupe River, Alviso Levees" dated 3-12-65.
- Existing Buildings
- Surface Seepage Observed on 12-2-82

SITE AND EXPLORATION PLAN
Guadalupe River Improvements
San Jose, California

Project No. 15482-V
Woodward-Clyde Consultants

Figure 1

N O T E S:

1. SEE FIGURE 1 FOR SECTION AND BORING LOCATIONS
2. THIS IDEALIZED SOIL PROFILE HAS BEEN CONSTRUCTED BY DIRECT INTERPOLATION BETWEEN BORINGS DRILLED AT VARYING SPACINGS AND PROJECTED TO THE SECTION LINE. THE DASHED LINES CONNECTING THE VARIOUS LAYERS AT EACH BORING LOCATION ARE FOR SCHEMATIC ILLUSTRATION PURPOSES ONLY AND SHOULD NOT BE CONSTRUED TO REPRESENT THE ACTUAL CONDITIONS IN THE FIELD.
3. FOR DETAILED DESCRIPTIONS OF THE MATERIALS ENCOUNTERED IN EACH OF THE BORINGS DRILLED FOR THIS INVESTIGATION, SEE FIGURES 4 THROUGH 17 .
4. THE EXISTING GRADE INDICATED IS APPROXIMATE AND IS BASED UPON INTERPOLATIONS BETWEEN SPOT ELEVATION DATA SHOWN ON AN AERIAL PHOTO BASE MAP OF THE PROJECT AREA (1982) FURNISHED BY THE SANTA CLARA VALLEY WATER DISTRICT.

L E G E N D:

▨ FILL
VARIO RES OF CLAYS, SANDS AND ROCK FRAGMENTS WITH
INCLU BROKEN ASPHALT, CONCRETE, WOOD AND OTHER DEBRIS.

▩ BAY
SOFT, SIBLE SILTY CLAY WITH TRACES OF ORGANIC MATERIAL.

▥ CLAYS
GENER F, MEDIUM TO HIGH PLASTICITY SILTY AND SANDY
CLAYS

░ SANDS
GENER IUM DENSE, FINE TO MEDIUM GRAINED SANDS AND
SILTY Y SANDS WITH SOME SCATTERED FINE GRAVEL.

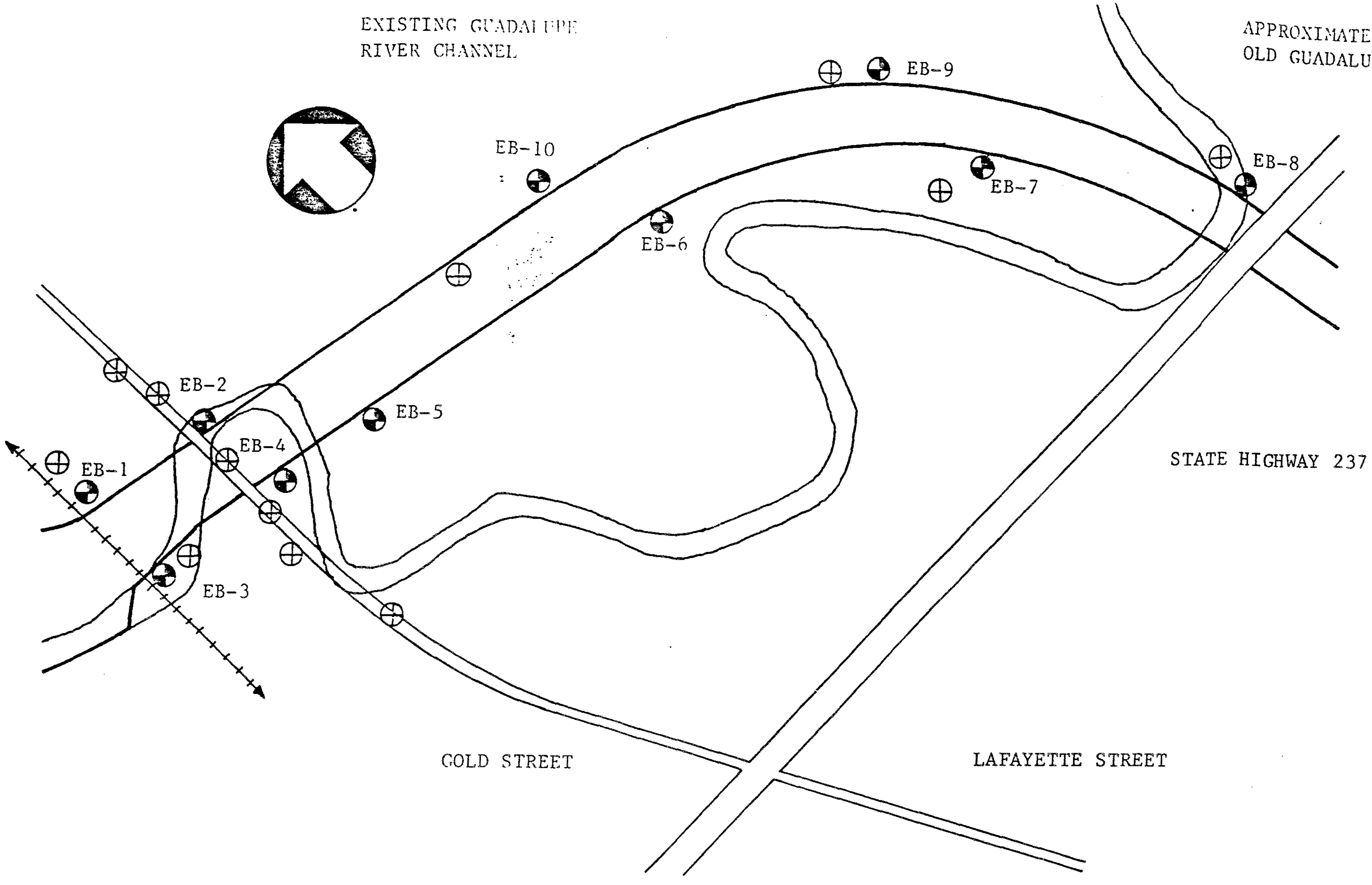
▽ HIGHEST WATER LEVEL RECORDED

0 50 100 200 300
HORIZONTAL SCALE IN FEET

VERTICAL EXAGGERATION: 10X

SECTION A - A: IDEALIZED SOIL PROFILE
GUADALUPE RIVER IMPROVEMENTS
San Jose, California

Project No. 15482V
WOODWARD-CLYDE CONSULTANTS
Figure 2



SCALE

0 100 200 300 400 500 feet

LEGEND

EB-1 ◕ Approximate location of exploratory boring

⊕ Approximate location of previous exploratory boring by others

John V. Lowney & Associates

Foundation / Soil / Geological Engineers

SITE PLAN: SPRR to HWY. 237

GUADALUPE RIVER IMPROVEMENTS
San Jose, California

PROJECT NO.	DATE	Figure 2
109-11	November 1980	